CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

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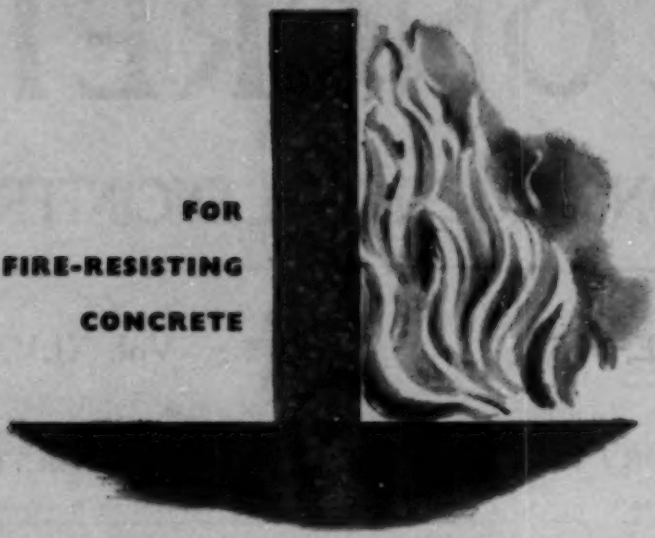

FOR
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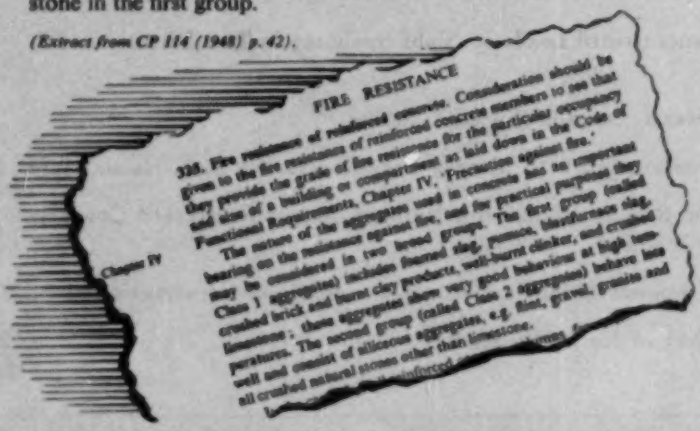
LIMESTONE AGGREGATE

The extract from the British Standard Code of Practice CP 114 (1948) shown below gives clear-cut information on the fire-resistance of reinforced concrete. It divides the nature of aggregates for practical purposes into two broad groups. The first group gives better results than the second. Limestone is the only natural stone in the first group.

(Extract from CP 114 (1948) p. 42).



Chapter IV

FIRE RESISTANCE

328. Fire resistance of reinforced concrete. Consideration should be given to the fire resistance of reinforced concrete members to see that they provide the grade of fire resistance for the particular occupancy and size of a building or compartment as laid down in the Code of Functional Requirements, Chapter IV, 'Precaution against fire.'

The nature of the aggregates used in concrete has an important bearing on the resistance against fire, and for practical purposes they may be considered in two broad groups. The first group (called Class 1 aggregates) includes foamed slag, pumice, blastfurnace slag, crushed brick and burnt clay products, well-burnt clinker, and crushed limestone; these aggregates show very good behaviour at high temperatures. The second group (called Class 2 aggregates) behave less well and consist of siliceous aggregates, e.g. flint, gravel, granite and all crushed natural stones other than limestone.

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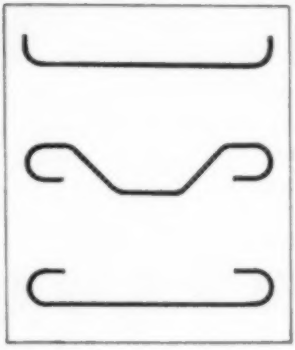

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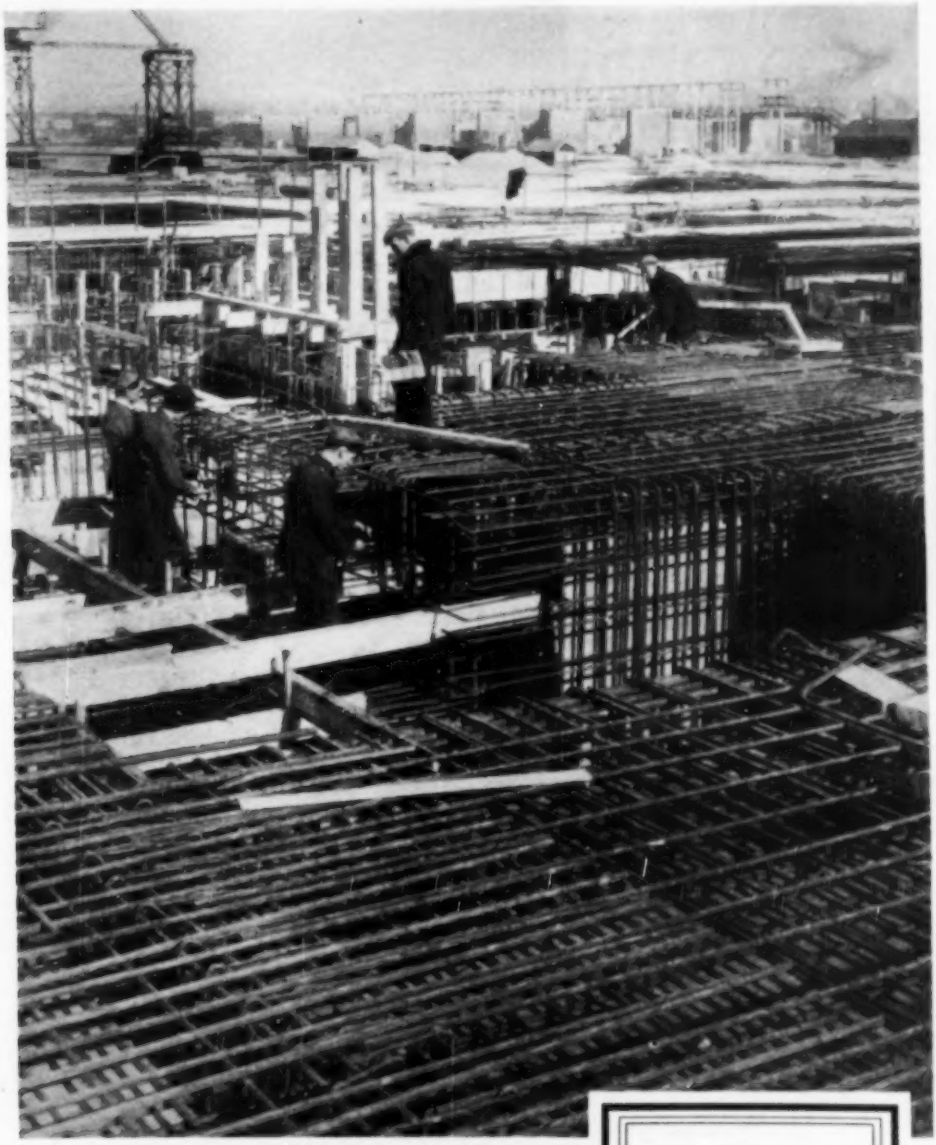

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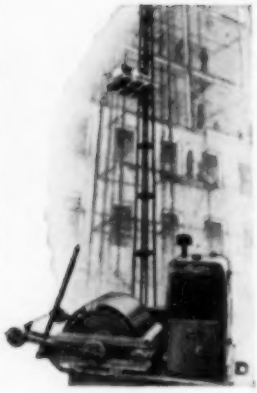

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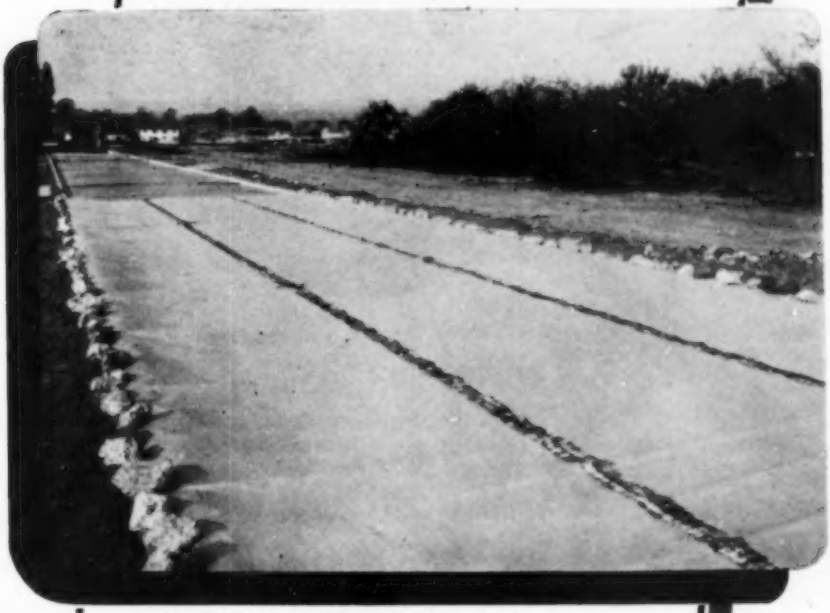

Showing Sisalkraft Concrete Curing Blankets as used on the Shepway Farm Housing Estate, Maidstone. Photograph by courtesy of John H. Hughes, Esq., M.C., M.I.Mun.E., F.R.San.Inst., Borough Surveyor. Contractors: Tilbury Construction Co., Ltd.

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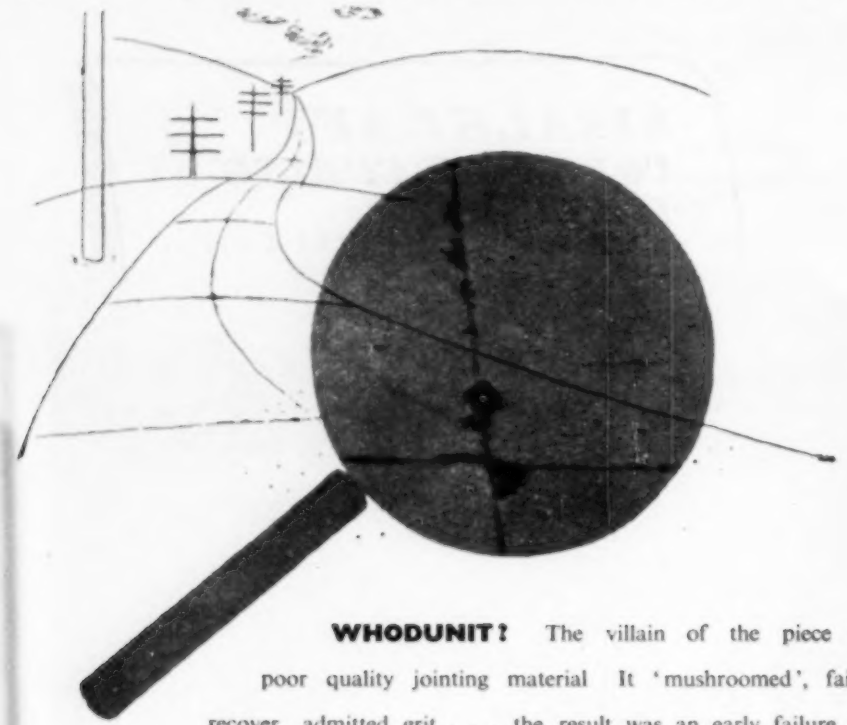

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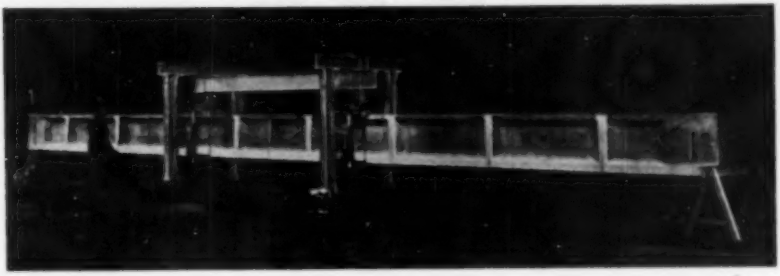

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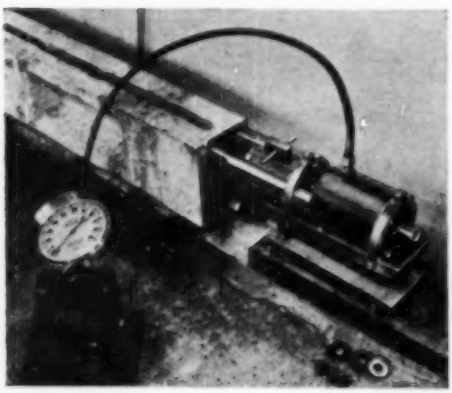

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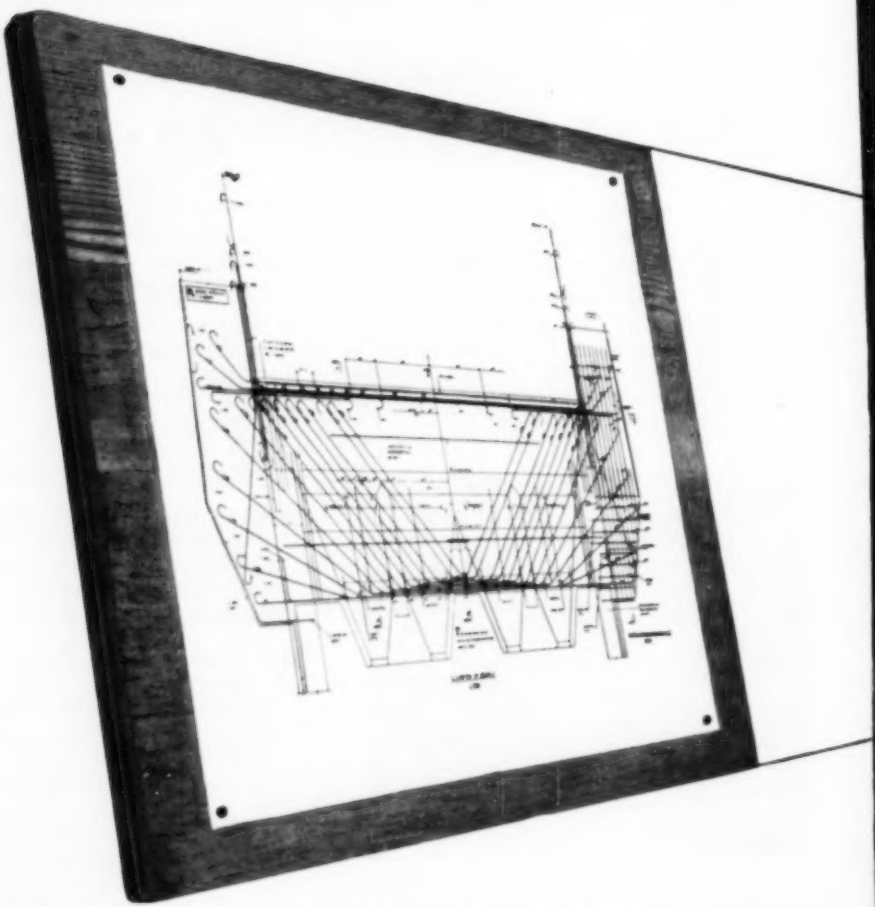

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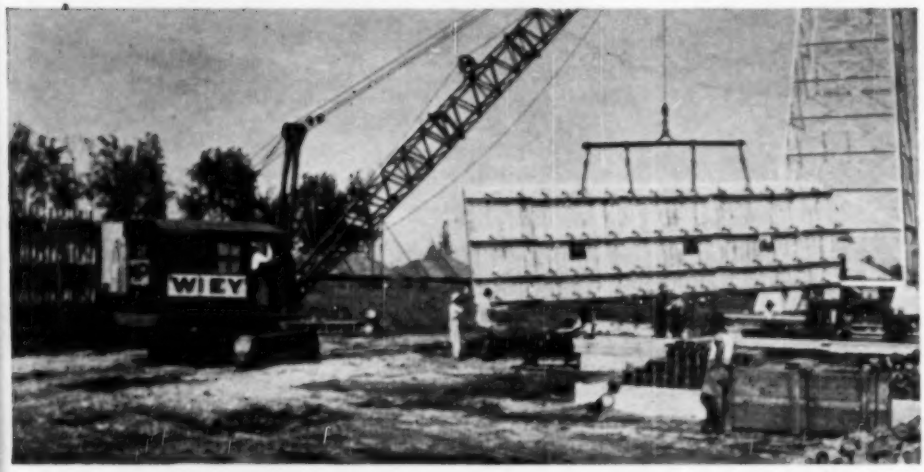

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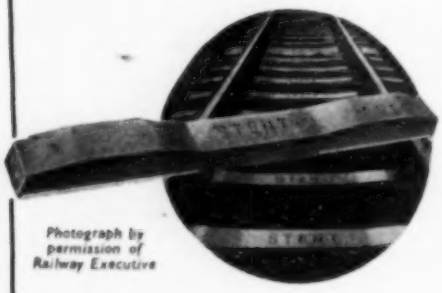

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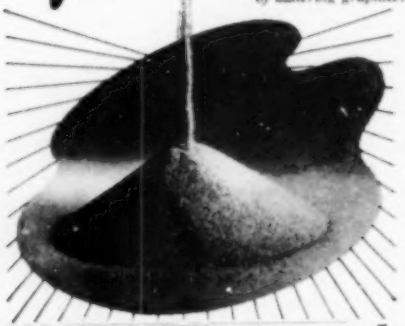

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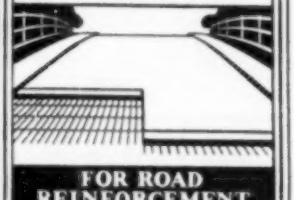

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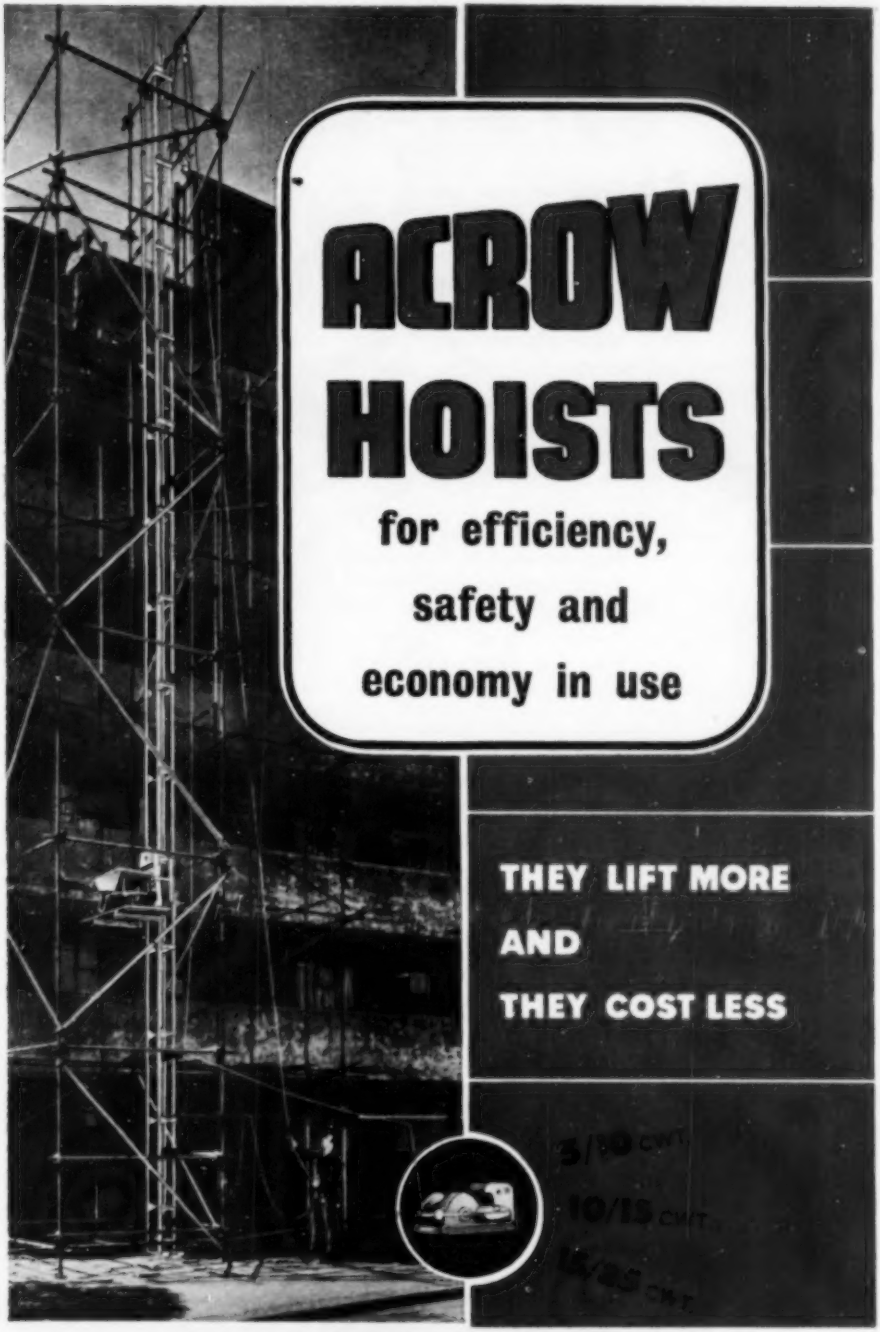

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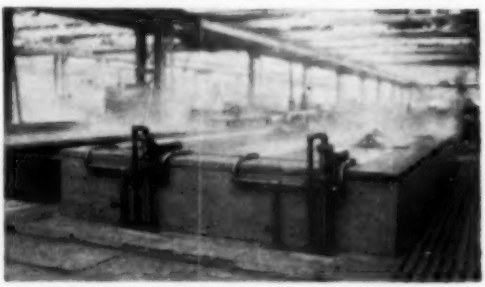

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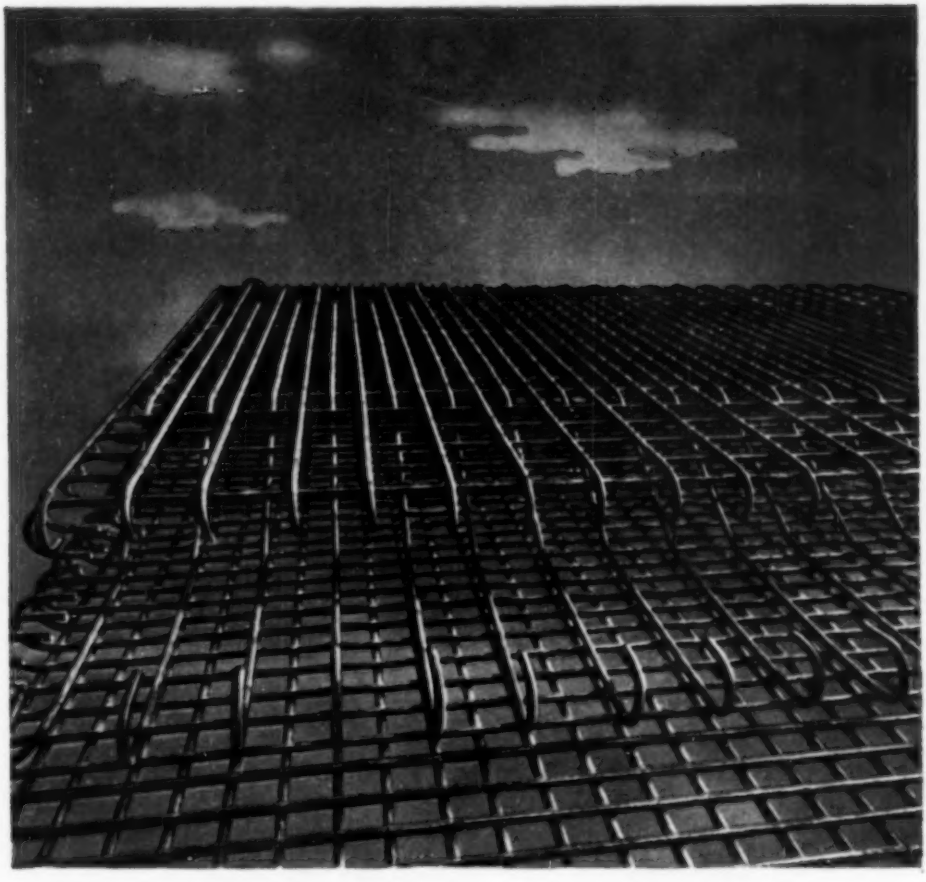

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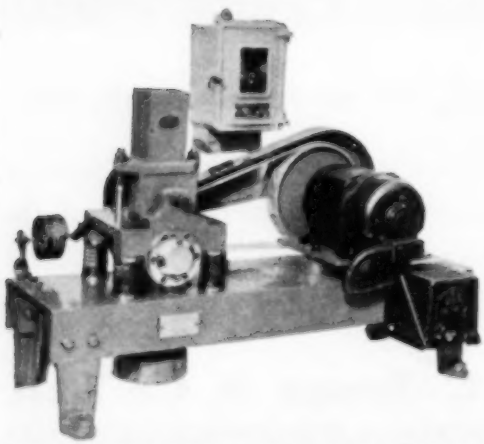

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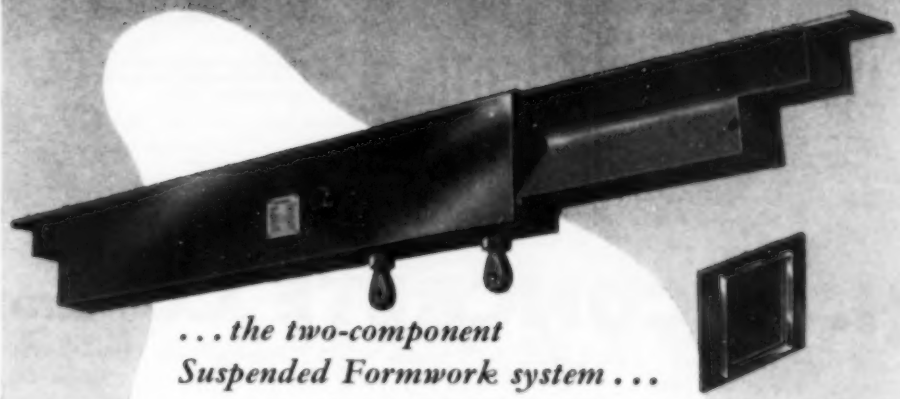

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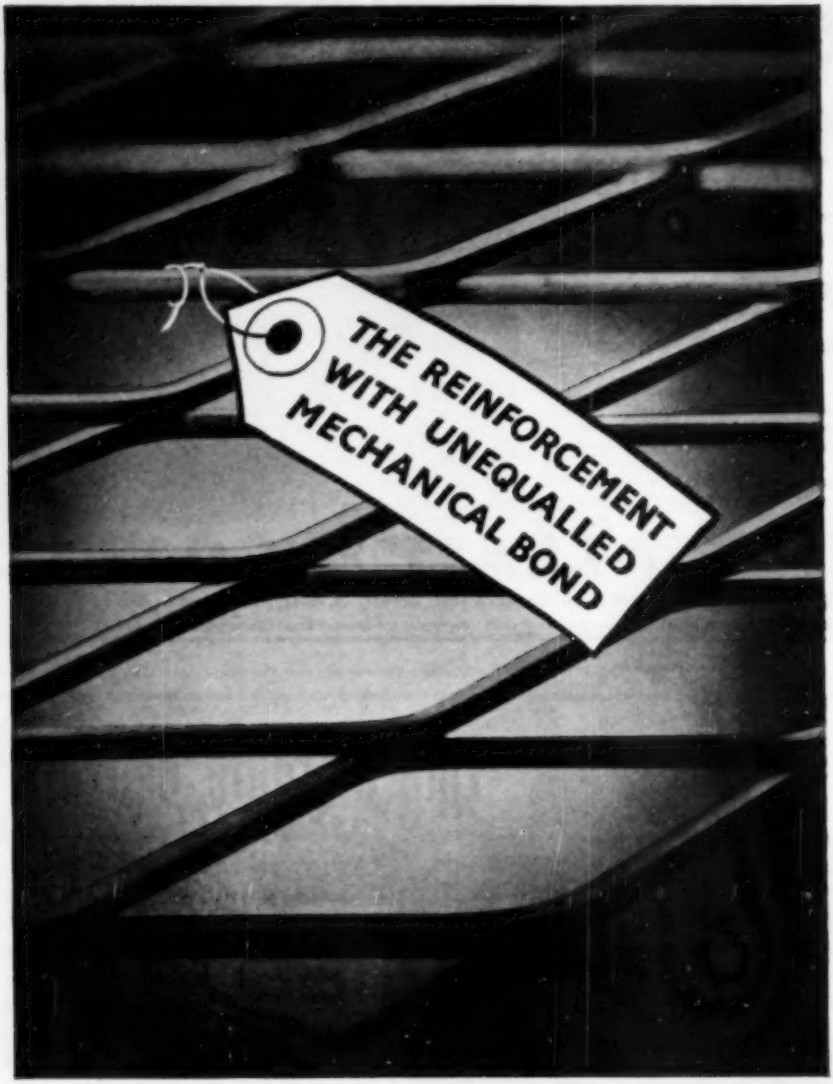

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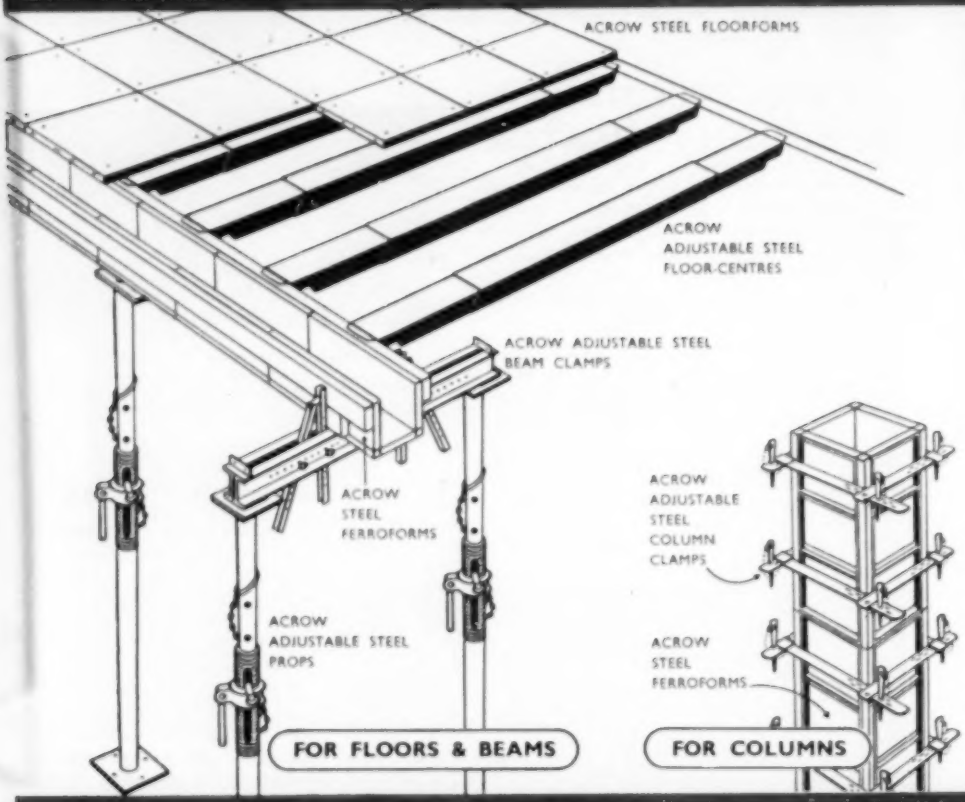

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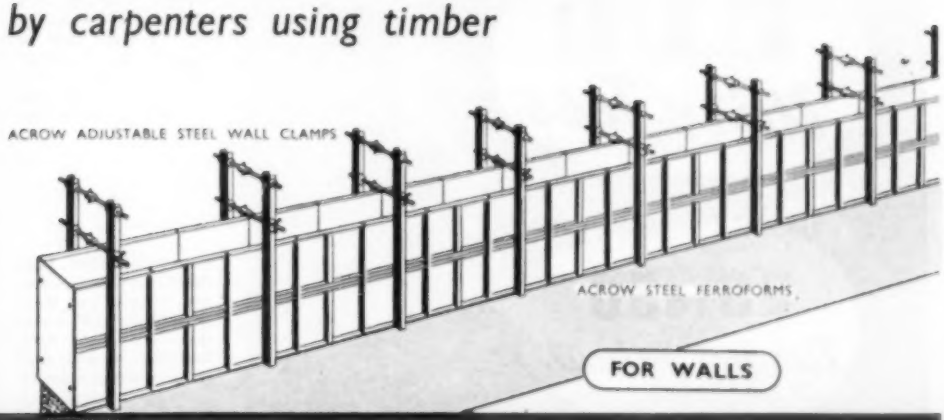

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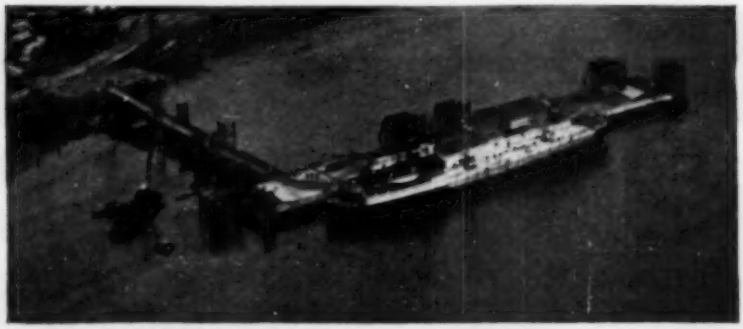

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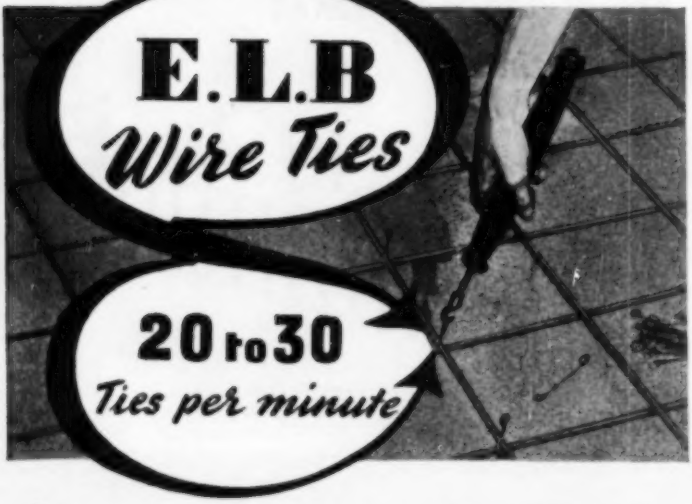

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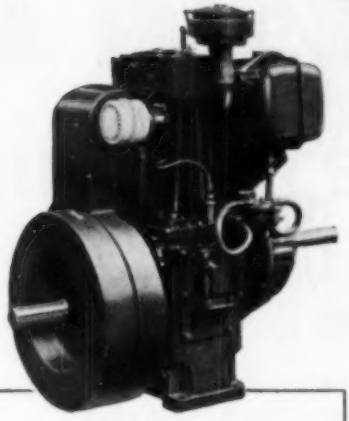

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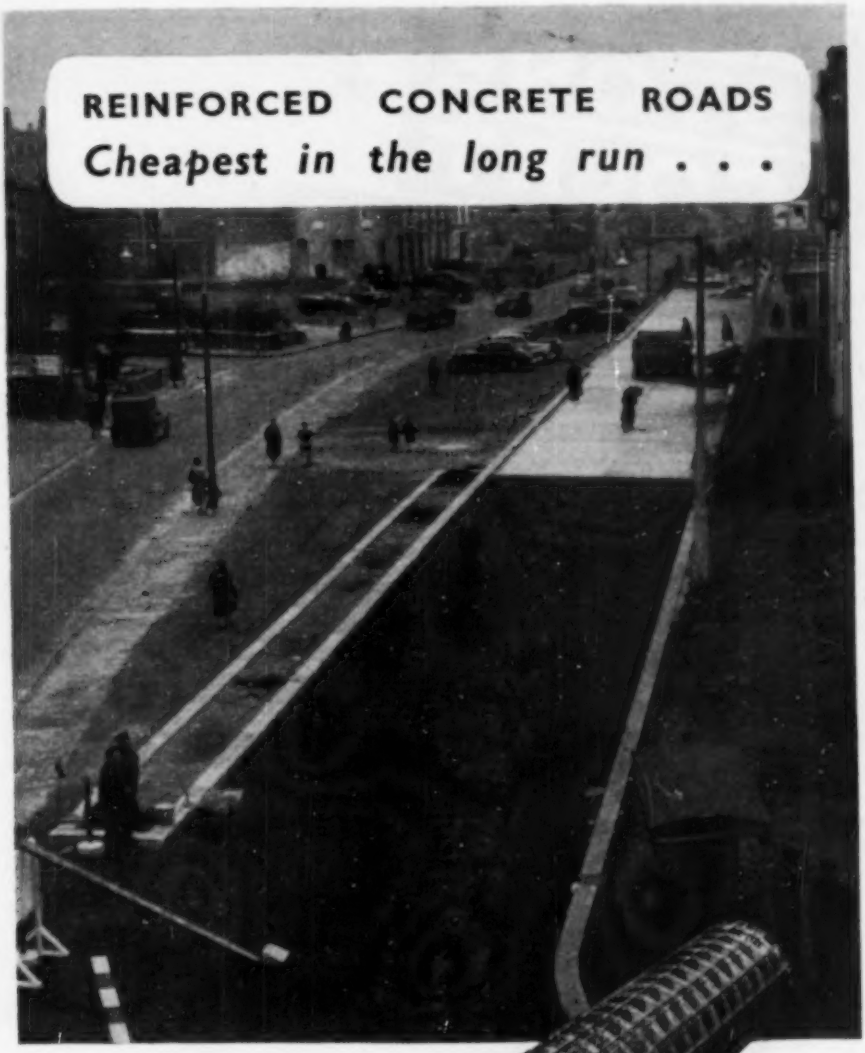

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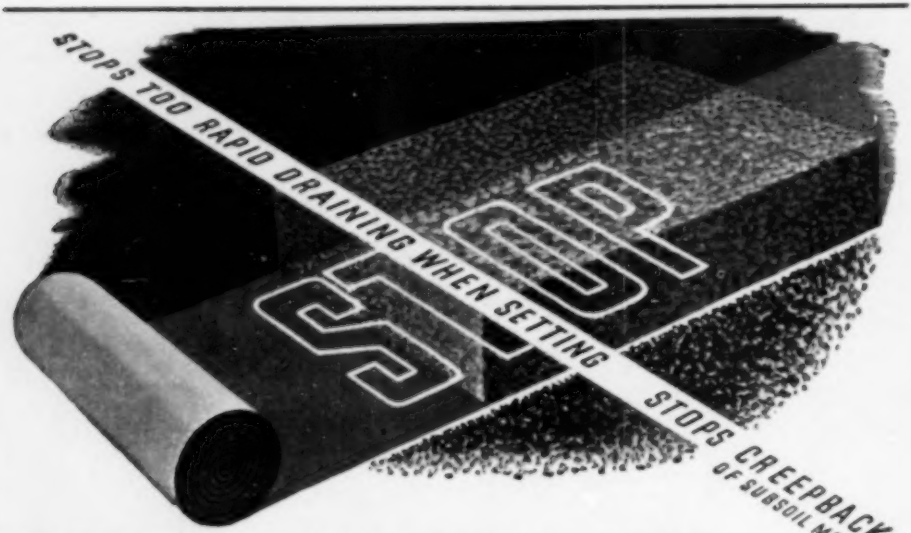

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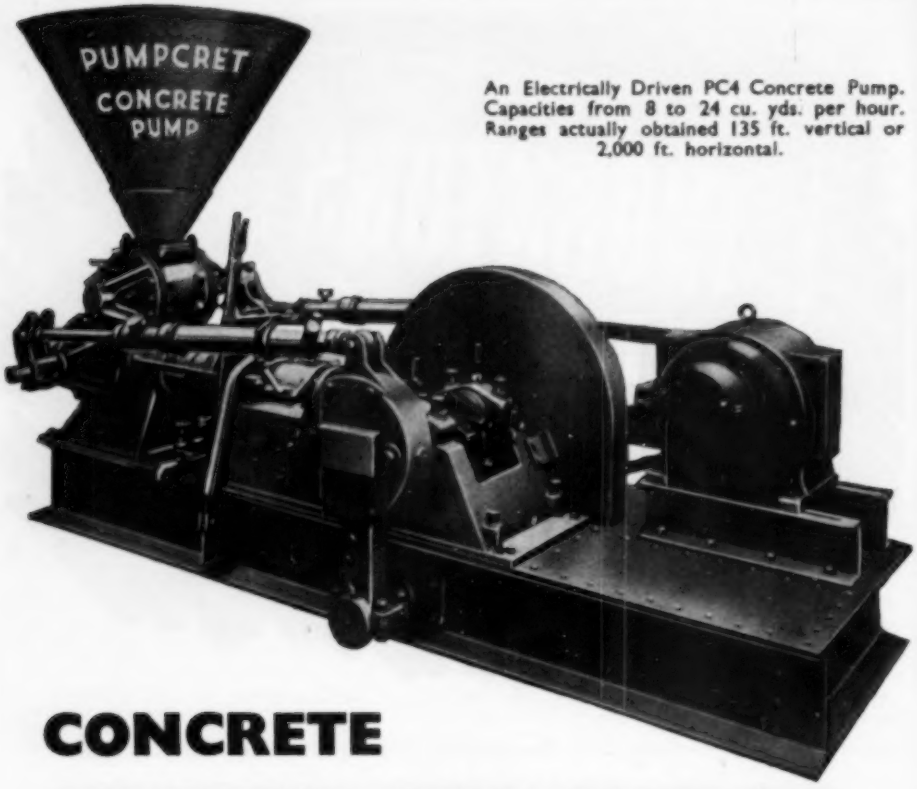

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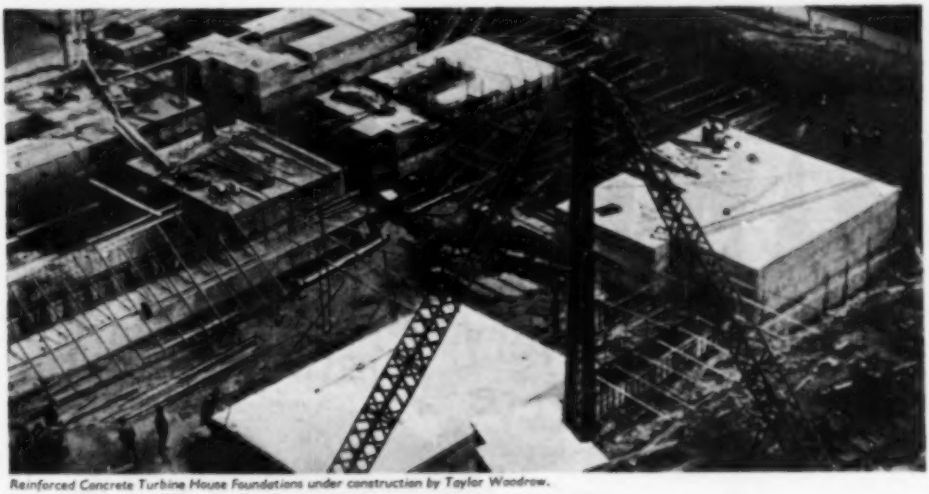

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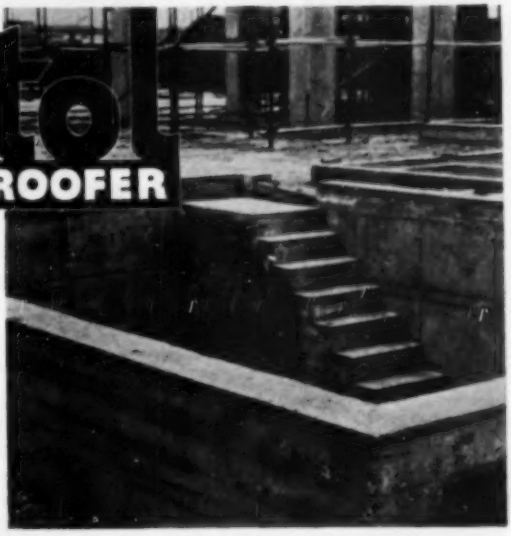

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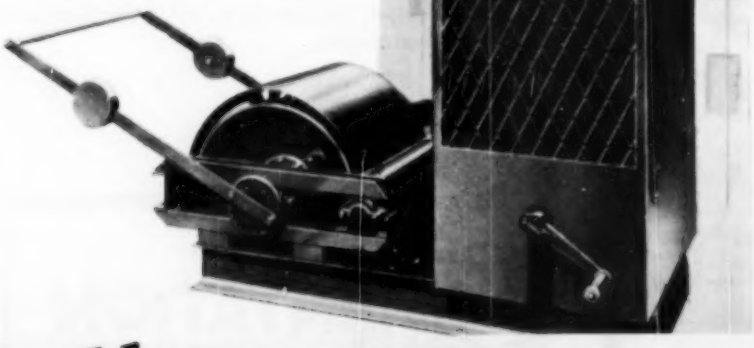

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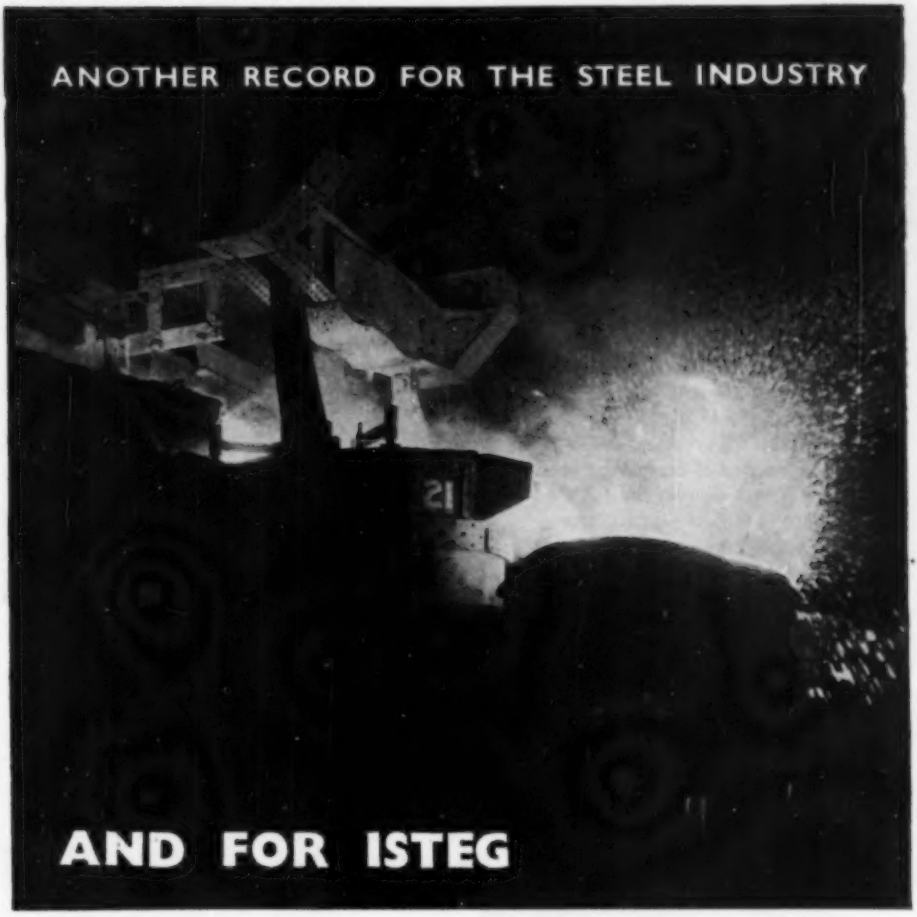

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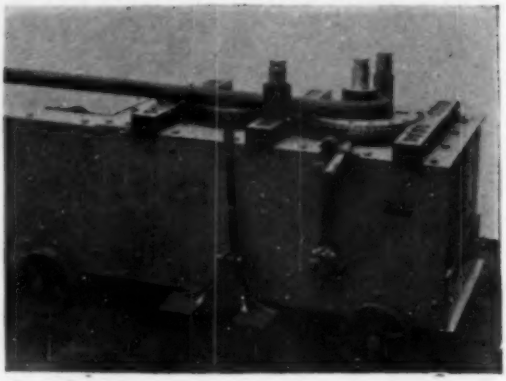

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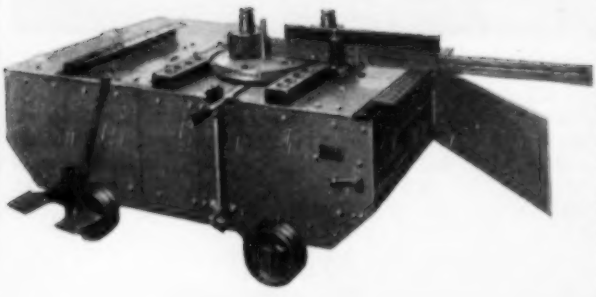

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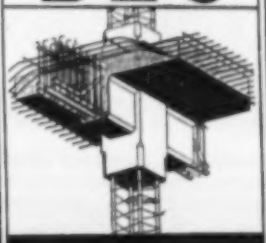

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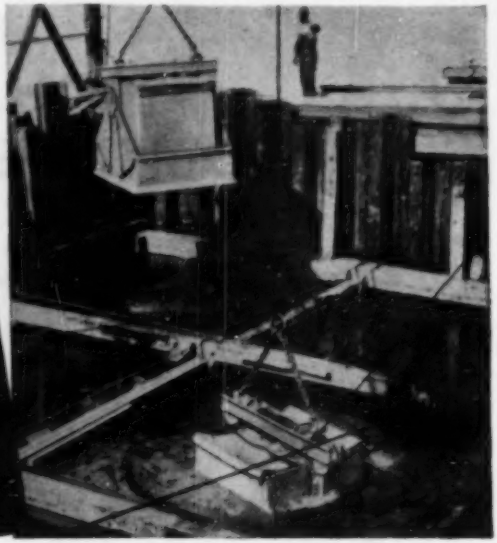

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XLV. No. 6. LONDON, JUNE, 1950

EDITORIAL NOTES

The Output of the Building Industry.

SELDOM have we read two such contrasting documents as the Report of the Building Industry Working Party (H.M.S.O. Price 2*s.* 6*d.*) and the Report of the Productivity Team representing the Building Industry (London : Anglo-American Council on Productivity. Price 2*s.* 6*d.*), published within a few days of each other last month. The Working Party deals with the building industry in Britain. The Productivity Team visited the United States of America, and its report is a glowing and stimulating account of an industry working at top speed and in which all concerned, from the architect to the labourer, have in view only the completion of the work in the shortest possible time. The following sentences from these two reports show the different outlook on production in the two countries.

According to the Working Party, in Britain, compared with the year 1937, the number of man-hours required to do the same amount of work had risen by 30 per cent. in 1948. After the war, emphasis was no longer laid on speed of construction rather than cost. It is claimed that the Essential Work Order which was in force between 1941 and 1947 left its mark on the industry in the form of relaxed discipline. The payment-by-results incentive scheme of 1941 is said to have affected the post-war output adversely ; operatives, so it is said, became disposed to regard the basic rates as indicators of a reasonable output in normal times, whereas in fact these rates were merely the result of a realistic appreciation of the exceptional circumstances then existing. Immediately after the war a vast programme of building work was launched without adequate planning, and the quantity of work started was only distantly related to the supply of materials and labour available. Scarcity of building materials has contributed perhaps more than any other single factor to the fall in productive efficiency. In a period of full employment a reserve of labour has been absent. Full employment has also influenced the geographical mobility of labour. Less competent workers have (under a policy of full employment) been able to remain continuously at work. The less efficient employers have had little difficulty in getting work. The security which the building operatives have enjoyed since the war has certainly tended to reduce the efforts of those among them who were formerly kept up to the mark by fear of unemployment. The building industry is once more affected by the cuts in capital investment required by the Government following the recent devaluation of sterling. The duplication of the technical examination of proposals

by officials employed by the various authorities is one of the reasons for the shortage of staff from which architects and surveyors suffer. Among its conclusions and recommendations the Working Party states that the industry must adjust itself to a policy of full employment and the absence of the reserve of labour previously available ; that the present arrangements for the issue of licences and permits cause delays and uncertainties which make planning impossible ; and that incentives are essential if output is to be adequately increased.

According to the report of the Productivity Team, in the U.S.A., now that adequate supplies of building materials and labour are assured, production per man-hour is 50 per cent. higher than in Britain on comparable types of building. "You have only to lean out of any mid-town window to notice the furious concentration and energy of construction workers while they are on the job. At five o'clock they will quit like an exploding light bulb, but up to that moment they haul and hammer and drill and bull-doze with fearful zest." The wages of labourers are about 65 per cent. of the craftsmen's wages [in Britain they are about 85 per cent.]. Building trade wages are amongst the highest in the U.S.A., and no doubt attract a high grade of labour. The high rates of wages, coupled with the consumer goods and other amenities which the worker is able to buy with his money, serve as incentives to output. There is also apparently a more serious risk of unemployment. The trades unions co-operate in a continuous process of designing and trying-out new methods to produce a better product at lower cost. The building operative works hard in return for a high wage, which provides him with a high standard of living, and knows that his personal advancement depends upon his own efforts. No one questions that the profit motive is an essential feature of industrial life. Site welfare is almost non-existent. The system of payment by results is not accepted by the trades unions and is unknown in contracting. A large part of the difference between American and British productivity can be accounted for only by the individual attitude towards work. In America the worker believes that by hard work only can he obtain the maximum financial benefit for himself ; he has never acquired the habit of doing less than he is capable of doing. The employer is recognised as entitled to his profits ; the larger the profit a firm makes the greater is the desire of its employees to continue to be associated with it.

Both of the reports are signed without reservation by the building trades union officials as well as by the architects, surveyors, and builders who comprised the Working Party and the Productivity Team. The absence of a civil engineer or a structural engineer in the Productivity Team is noticeable in some parts of this report and emphasises the suggestion made in the report that architects should have a greater knowledge of construction than is now common. There is little criticism of the professions and the builders except on the ground that there is less planning in Britain than in the U.S.A., but both agree that planning is impossible in Britain with the present controls and difficulties in getting materials and labour. Neither report faces the implications of what is called full employment, or differentiates between full employment and full useful employment. For example, is there not 24 per cent. of real unemployment in the British building industry when it takes 130 men to do what 100 did in 1937 ? Is there not real unemployment in the architectural and surveying professions when so many architects and surveyors are employed by Government departments and local

authorities in the duplicate checking of drawings, often produced originally by men whose knowledge of the work and the regulations is greater than that of the checkers? Also, it is clear that the Government looks upon construction as a "tap" that can be turned on and off for economic reasons—how in these circumstances can full employment be maintained except by all doing less when the tap is turned off? Or, when a planner decides that the volume of building work should be reduced, is it thought that, say, a plasterer should at once become a miner and a painter an agricultural worker, and that they should return to their proper trades when it is decided that there shall be more building? May we not even at this date question whether the popular idea of "full employment," which keeps on the payroll the slackers and the inefficient as well as the good workers, is better than the system by which, according to the report of the Productivity Team, only the best workers are kept in the industry? It seems that the men in the U.S.A. building trades work on an average forty weeks in a year at a wage of £25 a week or more and are able to save enough to add to the unemployment pay of £3 5s. to £6 5s. a week for single men to see them comfortably through a period of unemployment—and prefer to keep this system. That is, the money they earn in 40 weeks buys more commodities and amenities than does the British worker's earnings in, say, 50 weeks a year during which he produces less than the American. High wages for those who deserve them, with savings and unemployment pay to tide them over slack periods, seems a much better policy, for the State as well as for the men themselves, than continuous employment at less wages for all who can manage to become members of a building trades union, with a consequent higher rate of unemployment when there is less work because the Government has "turned off the tap" or for any other reason. It must be remembered, too, that there would be fewer delays due to lack of materials if people in some of the building materials industries produced more.

The term "full employment" seems to be much misused. In their ordinary sense the words mean a state where everyone is fully and usefully employed. In their political sense they seem to mean a state where everyone is attached to an organisation which pays him a wage, whether or not the wage is fully earned or whether the work done is useful; that is, people who are paid for unnecessarily checking work that has already been checked and people who are paid for idling on a building site are now said to be in full employment. Both reports emphasise the need for more pay for more work, and if this can be achieved it is the best solution of the problem of low output. But in our opinion it is important that the extra pay should be really worth striving for, not only in the sense that a good worker should have a high wage but that the difference between the wages of a good worker and a slacker should be such that the slacker should have a real incentive to work harder and earn more—a difference of a few shillings a week only between the earnings of a hard-working craftsman and a lazy labourer is more likely to encourage the hard worker to become a slacker.

The word "pre-planning" occurs many times in the report of the Productivity Team. It probably means planning, for what is done before planning starts does not seem to be of much importance.

Concrete Roads and Runways.

Stresses in Concrete Roads and Runways.

In Proceedings No. 13 of the Swedish Cement & Concrete Research Institute ("Investigations of Wheel Load Stresses in Concrete Pavements", by S. G. Bergström, E. Fromen, and S. Linderholm. Stockholm. 1949. In English. Price 12 kr.), consideration is given to the many theories and test results available relating to the stresses in concrete slabs on the ground and subjected to wheel loads. The effects of other factors giving rise to stresses are disregarded. Tests over short and long periods and to failure were made by the authors on full-size slabs with the object of comparing theoretical and actual stresses, principally in runways.

An important result of the tests is that the distribution of pressure on the ground due to a load applied for a comparatively short time is in good agreement with theories based on the assumptions that the slab and the ground are elastic, isotropic, and homogeneous, and that the ground acts as a semi-infinite body. It was not possible to study the effect of plastic deformations of the ground on the distribution of pressure but, as the load increased, a marked increase in pressure under the middle of the slab immediately before failure was evident. The deflections calculated by the elastic theory of soil are in satisfactory agreement with those obtained in the tests. The critical load calculated by the elastic theory of soil was about 80 per cent. of the load at failure, but the curves of pressure distribution and deflection indicate that failure started at a load of about 95 per cent. of the load at failure. The difference between the calculated and the observed critical loads is probably due to plastic equalisation of stresses which is not taken into account in calculations based on the elastic theory.

The system used for the pressure measurements was satisfactory for the short-time tests and the test to failure, but not for the long-time tests in which disturbance was caused by the variable warping of the slab due to fluctuations of the air temperature. The sensitivity required of the apparatus was very severe as the allowable deformation was only 1 micron at a pressure of 100 gr. per square centimetre. A load of 1 ton resulted in a mean pressure of about 5 gr. per square centimetre and the load at failure on a 10-in. slab of about 20 ft. diameter was about 40 tons.

Temperature Stresses.

In Proceedings No. 14 ("Temperature Stresses in Concrete Pavements," by S. G. Bergström. Printed in English. Price 4 kr.) a theoretical investigation leads to methods, formulæ, and diagrams for the calculation of temperature stresses in concrete roads and runways. Special attention is given to the tensile stresses in the bottom of the slab, as they are of particular interest in the design of unreinforced slabs and slabs reinforced to prevent cracking due to secondary stresses. The results can be used for determining the theoretical distance between warping joints and between contraction joints. The tensile stresses in the top of the slab are also considered as they are of importance in the design of structurally-reinforced slabs. A theory is also described for determining the greatest permissible distance between expansion joints to prevent the compressive stresses causing the slab to lift off the ground. The theoretical stresses are compared with the stresses observed in tests and agree reasonably closely. The coefficient of friction assumed is also based on tests, and applies to a relatively smooth foundation such as is considered desirable in practice. The maximum coefficients vary from 3·1 for a 4-in. slab to 2·3 for an 8-in. slab.

A Prestressed Concrete Continuous-girder Bridge in Belgium.

THE continuous-girder bridge of two spans (*Fig.* 1) recently constructed across the river Meuse at Sclayn, near Namur, Belgium, is a prestressed concrete structure in which the Magnel-Blaton system is used. The pier and abutments of a previous bridge on this site were used to support the new superstructure.

I.—DESIGN AND CONSTRUCTION.

The bridge is designed and constructed to comply with the requirements of the bridges and roads authority of the Belgian government, the standard load for road bridges being a moving load of 32 tons and a uniformly-distributed load of 82 lb. per square foot.

The two main spans (*Fig.* 2) are each 206 ft. 9 in. long. The roadway is about 23 ft. wide and there are two cantilevered footpaths each about 5 ft. wide. The overall width of the bridge is about 33 ft. 10 in., the cross section being cellular and comprising four longitudinal walls and a transverse wall at each of the third points of each span. Rectangular openings are provided in the transverse walls for inspection and for the passage of the cables. The continuous beams are supported at the middle pier on a hinge of the Freyssinet type, and at the outer

Fig. 1.—Sclayn Bridge.

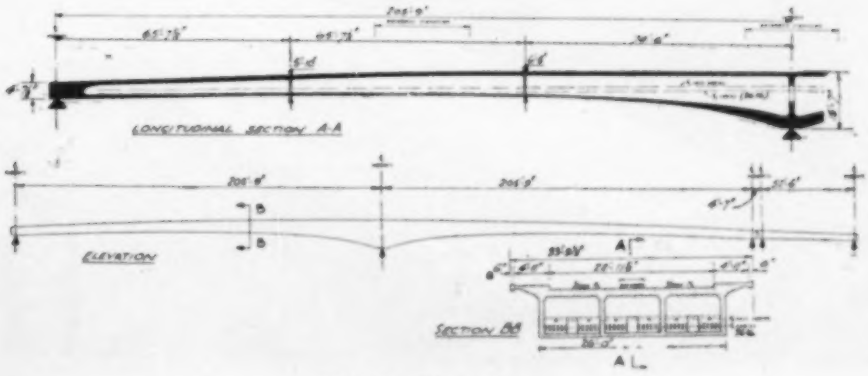


Fig. 2.—Dimensions.

supports of each span reinforced concrete rockers are provided. The approach span at one end of the bridge comprises 16 simply-supported precast prestressed concrete beams.

The main beams are prestressed by 36 cables each of which contains 48 wires of 7 mm. (0·276 in.) diameter. The cables are distributed uniformly across the width of the bridge and extend the full length of the structure, the distance between the anchorages being about 416 ft. The cables are straight in each span and change direction slightly at the middle pier. The anchorages are sandwich-plates bearing on plates which distribute the force on to the concrete. In each of the suspended beams in the approach span there are two cables each containing 24 wires of 7 mm. diameter. After these beams had been erected, they were prestressed transversely by ten cables each of which contains eight 7-mm. wires.

Centering and Concrete.

The old pier and abutments were first reconditioned to suit the new superstructure, the timber shuttering for which was supported on temporary centering of steel scaffold tubes (*Fig.* 3) erected on a platform carried on timber piles. Over a temporary navigable span the shuttering was carried on steel beams supported at each end on a group of piles. The steel beams over the navigable span deflected excessively and caused a crack in the top of the concrete structure near the temporary piers formed by the groups of piles. The crack closed and became invisible when the prestressing operations were completed.

The first stage in the construction was to form the concrete hinges at the middle pier and abutments. The soffit, or intrados, slab was then concreted commencing at the abutments and proceeding towards the middle of each span. The longitudinal walls and the end-blocks of beams were concreted next and the cables were laid in position (*Fig.* 4). The transverse walls were then concreted and

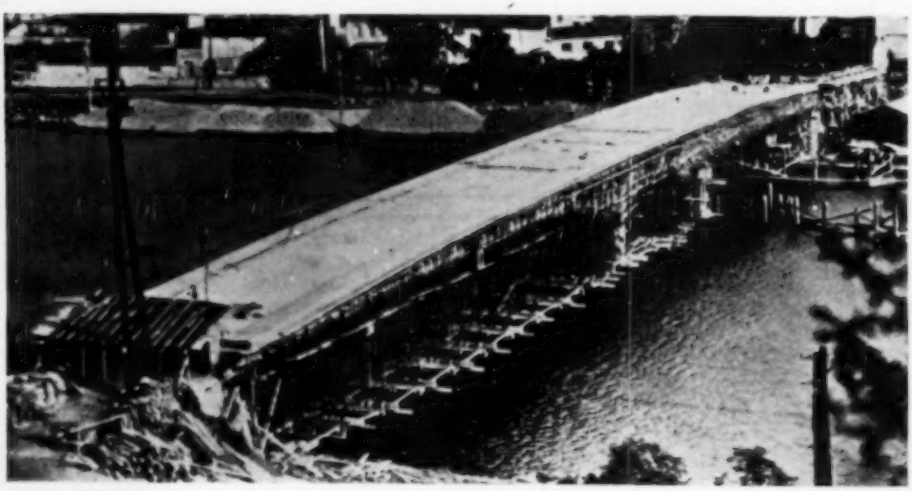


Fig. 3.—Centering.

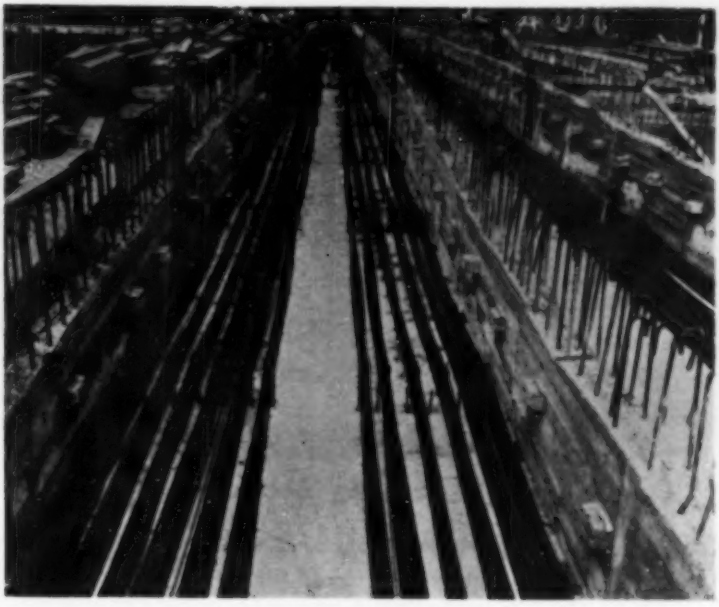


Fig. 4.—The Prestressing Cables.

this operation was followed by concreting the road slab and footpaths in a similar manner to the soffit slab. The concrete was mixed on the east bank of the river and transported in two-wheel skips pulled by hand along an elevated gangway erected along the centre line of the bridge.

The permissible compressive stress in the concrete is 2160 lb. per square inch and the permissible tensile stress is 110 lb. per square inch. There are 1215 cu. yd. of concrete, 42 tons of mild steel reinforcement, and 26 tons of high-tensile wire of 7 mm. diameter.

Prestressing.

The cables were stretched (*Fig.* 5) as soon as tests showed that the concrete had attained sufficient strength. About 100 wires were stretched in a day, two wires being stretched at a time. Owing to the exceptional length of the wires, two jacks, one at each end of the bridge, were used for each pair of wires. The arrangement of the wires in the cables made the selection of the same wire by the jacking teams at each end of the bridge an easy matter. Most of the wires were stretched by jacks actuated by manually-operated hydraulic pumps, but mechanically-operated pumps, fitted with relief valves to control the greatest pressure when the required extension was obtained, were used at one stage of the operations.

As the cables lie in the cells between the longitudinal walls, it was not possible to protect them by grouting. Shuttering was therefore erected around the stretched cables which were then encased in fine concrete placed and consolidated by hand.

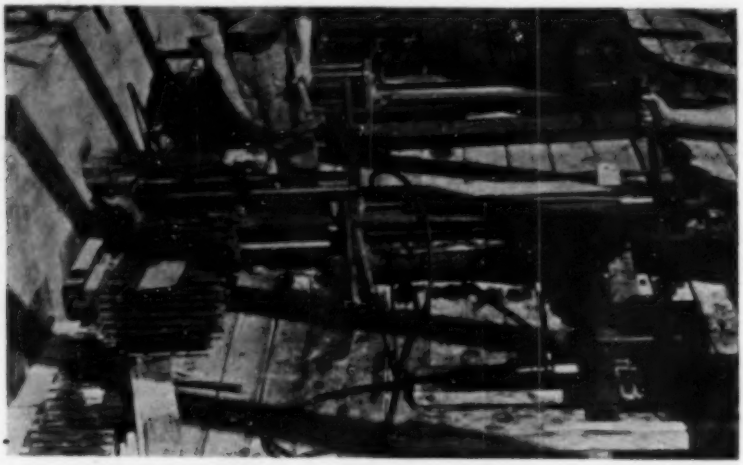


Fig. 5.—Stretching the Cables.

The consulting engineer for the bridge is M. Alexandre Birguer. The preliminary investigation and the supervision of the final design were made by Professor Gustave Magnel. The contractors are Entreprises Blaton-Aubert. Many designs and tenders were submitted for the construction of the bridge, but the tender accepted for the construction in prestressed concrete was about 8 per cent. less than tenders for construction in steel and 20 per cent. less than for reinforced concrete.

II.—DESIGN CALCULATIONS.

The calculations in the following were prepared by Professor Magnel and are based on the preliminary design of the bridge. The results do not quite comply with those relating to the structure as built, but are close enough for the purpose of showing the principles on which the structure is designed. A summary of the principal data follows.

The bridge is a continuous structure of two spans, the theoretical length of each being 205·72 ft. (62·70 m.). *Fig.* 6 shows the elevation and cross section of the structure, and gives the dimensions upon which the calculations are based. The prestress was induced by 36 cables each comprising 48 wires of 0·276 in. (7 mm.) diameter stressed at 121,000 lb. per square inch, giving an initial prestressing force of 12,456,000 lb. (5650 metric tons) reducing to 0·85 × 12,456,000 = 10,588,000 lb. (4800 metric tons) in course of time.

The position of the cable, which is in a straight line in each span, is shown in *Fig.* 6, and the resulting eccentricities are easily computed when the positions of the centroids at the various cross sections are known. *Table* I gives the useful properties of cross sections at ten positions spaced equally along each span. The span should be divided into, say, twenty parts in order to gain greater accuracy; but this is not done in this example because the reader would learn nothing more from it.

TABLE I.—GEOMETRICAL PROPERTIES OF THE BEAMS (*Fig.* 6).

Cross section	Depth		Centroid of section			Eccentricity of cable (e) (ft.)	Moment of inertia (I) (ft.^{4})	Area (A) (sq. ft.)	(Radius of gyration)^2 $r^2 = \frac{I}{A}$ (ft.^{2})
	At crown of road (ft.)	Maximum (ft.)	y_1 (ft.)	$y_{1(max.)}$ (ft.)	y_2 (ft.)				
0	4·62	5·51	2·03	2·92	2·59	1·08	284	129	2·20
1	4·85	5·74	1·77	2·66	3·08	1·97	255	67·2	3·84
2	5·25	6·14	1·67	2·56	3·58	2·69	251	61·6	4·09
3	5·74	6·63	1·74	2·63	4·00	3·28	278	60·5	4·62
4	6·07	6·96	1·80	2·69	4·27	3·58	305	60·5	5·08
5	6·36	7·25	1·87	2·76	4·49	3·64	335	60·5	5·59
6	6·56	7·45	2·13	3·02	4·43	3·25	394	63·7	6·24
7	7·05	7·94	2·49	3·38	4·56	2·79	545	68·0	8·08
8	8·10	8·99	3·05	3·94	5·05	2·16	780	72·3	10·8
9	10·89	11·78	5·22	6·11	5·67	— 0·03	2045	98·3	20·9
10	15·58	16·47	10·32	10·21	6·26	— 4·30	5612	157·7	35·9

NOTE.—y_1 is measured at the crown of the road. y_1 (*max.*) is at the handrail kerbs.

The notation used is the following:

A, cross-sectional area of beam.
c_b, c_t, calculated stresses in the bottom and top fibres respectively.
D, total depth of beam.
e, eccentricity of the cable from the centroid.
I, moment of inertia about the horizontal centroidal axis.
M, bending moment; M_{10}, M_5, etc., bending moments at points (10), (5), etc.
M_{A1}, secondary bending moment at middle support.
P_i, initial stretching force.
r, radius of gyration of the concrete section.

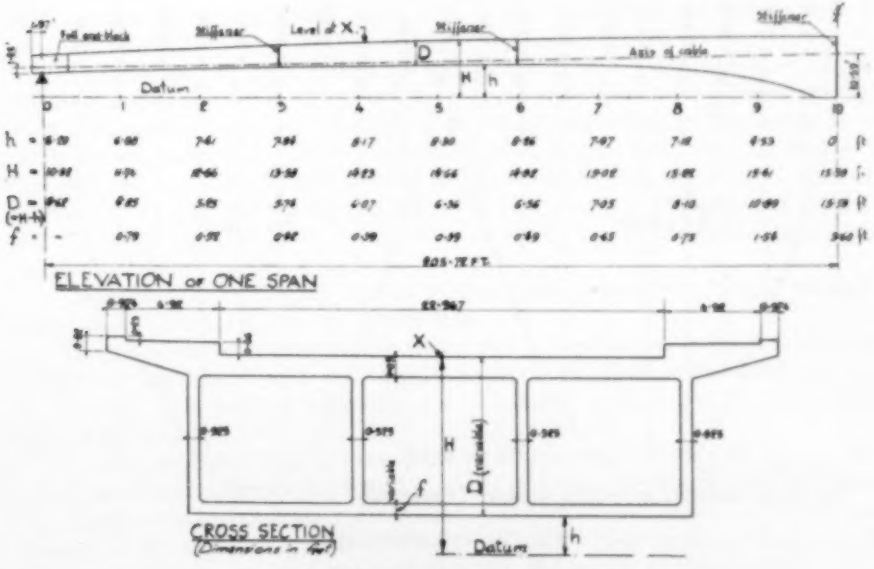


Fig. 6.—Dimensions.

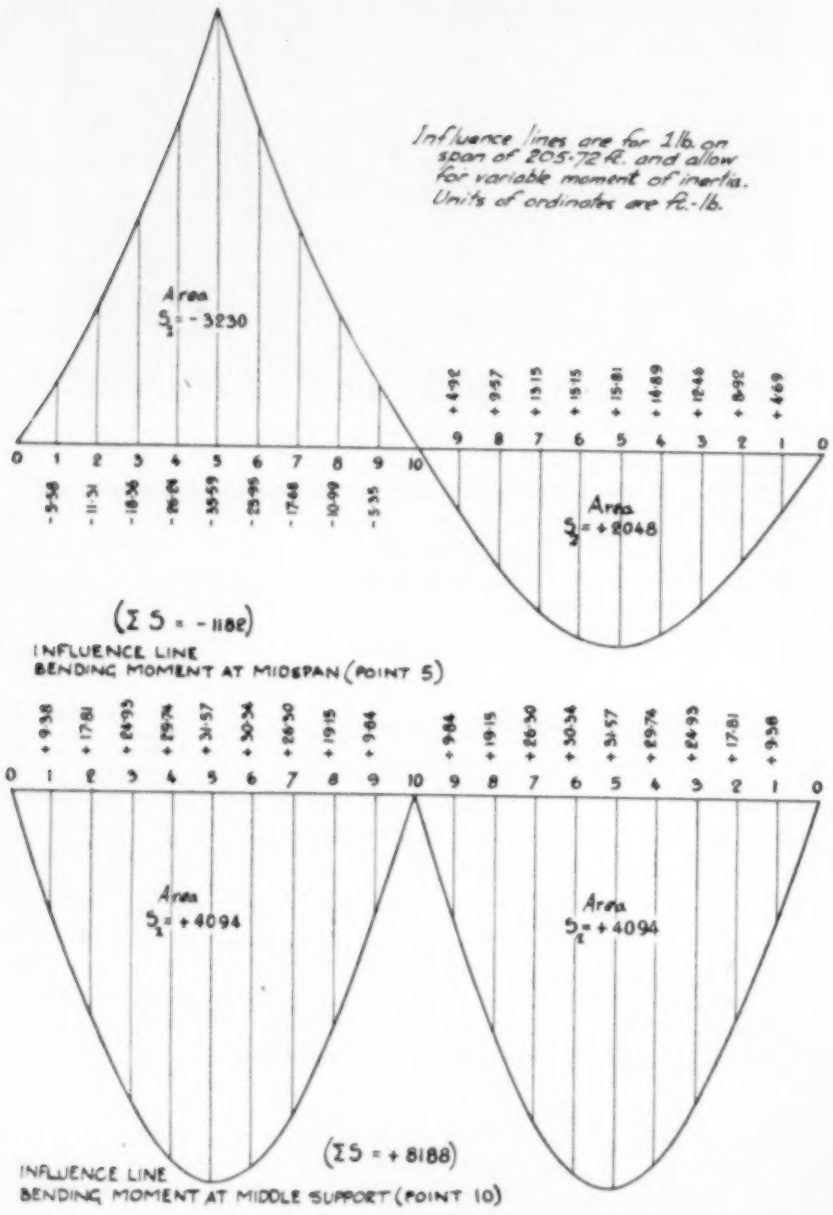


Fig. 7.—Influence Lines.

x_l, ratio of distance (from the left-hand support) of a point to the length of the span.
y_1, y_2, distances from the centroid to the top and bottom fibres respectively; y_1 is measured at the crown of the road, and $y_{1(max.)}$ at the handrail kerbs.
α, β, relate to conditions for maximum negative and maximum positive bending moments due to live load for point (5).

The loads are based on the data in the following. The dead load is based on the weight of concrete being 156 lb. per cubic foot (including the steel therein). The surfacing of the road weighs 36·1 lb. per square foot. There is no surfacing on the footpaths. The two handrails weigh together 67·2 lb. per linear foot. The weight of the fine concrete deposited around the cables is estimated to be 806 lb. per linear foot of bridge. The moving loads consist of a uniformly-distributed load of 82 lb. per square foot everywhere including footpaths, and a test-vehicle of about 32 tons comprising axle weights of one of 12 tons, two of 6 tons, and two of 4 tons; the spacing of the axles is 13·1 ft.; two test-vehicles are assumed to travel side by side.

Bending Moments due to Loads.

The first step is to concentrate the dead load at points 0, 1, 2, . . . 10 of each span, giving the following concentrated loads:

Point	0	1	2	3	4	5
Load (metric tons)	93·8	97·0	89·2	88·0	88·0	88·0
,, (lb.)	206,800	213,900	196,700	194,000	194,000	194,000
Point		6	7	8	9	10
Load (metric tons)		92·0	99·2	105·8	141·8	114·0
,, (lb.)		202,900	218,700	233,300	312,700	251,000

The total weight of each span is 2,419,000 lb. (1097 metric tons), excluding the stiffeners. Adding the weight of the stiffeners at two sections (3), 10,300 lb. (4·7 tons) each, at two sections (6), 12,100 lb. (5·5 tons) each, and at two sections (10), 47,400 lb. (21·5 tons) each, the total dead load including steel is $(2 \times 1097) + 2(4{\cdot}7 + 5{\cdot}5 + 21{\cdot}5) = 2236$ tons, or 4,930,000 lb.

It is now necessary to determine the ordinates of the influence lines for the bending moments at the central support, point (10), and at midspan, point (5). This is an ordinary problem presenting no peculiarity owing to the beams being in prestressed concrete, and it therefore suffices to give the diagrams in *Fig.* 7. The computation is restricted to cross sections (10), the middle support, and (5), the middle of the span, since the object of this example is purely pedagogical. For the design of the actual bridge it is necessary to do the calculations for a greater number of cross sections.

Using the influence lines in *Fig.* 7, the bending moments for the following cases (*a*), (*b*), and (*c*) can be determined.

Case (*a*).—When prestressing is completed. The bridge carries only the dead weight of the beams and stiffeners calculated as concentrated loads at 0, 1, 2, . . . 10. The bending moments are calculated as in *Table* II, from which M_{10} is 84,642,000 ft.-lb., and M_5 is 11,689,000 ft.-lb.

It must be noted that in these calculations positive bending moment is that which produces concavity downwards, and is therefore the reverse of the ordinary rule which considers concavity downwards (such as at the supports of continuous beams) to be negative. Also it should be noted that bending moments are expressed as total bending moments for the full width of the bridge.

TABLE II.—BENDING MOMENTS FOR CASE (*a*).

Loaded Point		Load (lb.)	Ordinate of influence line for M_{10}	B.M. at support (ft.-lb.)	Ordinate of influence line for M_5	B.M. at midspan (ft.-lb.)
Left-hand span	0	206,800	0	0	0	0
	1	213,900	9·38	2,007,000	5·57	− 1,192,000
	2	196,700	17·81	3,503,000	11·31	− 2,225,000
	3	194,000				
		10,300				
		204,300	24·93	5,097,000	18·36	− 3,751,000
	4	194,000	29·74	5,770,000	26·24	− 5,090,000
	5	194,000	31·57	6,124,000	35·59	− 6,905,000
	6	202,900				
		12,100				
		215,000	30·34	6,523,000	25·95	− 5,579,000
	7	218,700	26·30	5,752,000	17·68	− 3,867,000
	8	233,300	19·15	4,468,000	10·99	− 2,564,000
	9	312,700	9·84	3,077,000	5·35	− 1,672,000
Support	10	251,000	0	0	0	0
		251,000				
		502,000				
Right-hand span	9	312,700	9·84	3,077,000	4·92	+ 1,538,000
	8	233,300	19·15	4,468,000	9·57	+ 2,233,000
	7	218,700	26·30	5,752,000	13·15	+ 2,876,000
	6	202,900				
		12,100				
		215,000	30·34	6,523,000	15·15	+ 3,257,000
	5	194,000	31·57	6,124,000	15·80	+ 3,065,000
	4	194,000	29·74	5,770,000	14·86	+ 2,883,000
	3	194,000				
		10,300				
		204,300	24·93	5,097,000	12·46	+ 2,545,000
	2	196,700	17·81	3,503,000	8·92	+ 1,756,000
	1	213,900	9·38	2,007,000	4·69	+ 1,003,000
	0	206,800	0	0	0	0
Totals .	.			84,642,000		− 11,689,000

Case (*b*).—Construction completed but live load not acting.—To the bending moments calculated in (*a*) must be added those due to the additional dead load, namely

Road surface: 22·97 × 36·1 =	829	lb. per ft.
Handrails:	67	,, ,, ,,
Fine concrete around the cables:	806	,, ,, ,,
Total	1702	,, ,, ,,

From the areas of the influence-line diagrams, the bending moments due to the foregoing uniformly-distributed loads are

$$M_{10} = 1702 \times 8188 = 13{,}930{,}000 \text{ ft.-lb.}$$
$$M_5 = 1702 \times (-1182) = -2{,}010{,}000 \text{ ft.-lb.}$$

The total bending moments for case (*b*) are

$$M_{10} = 84{,}642{,}000 + 13{,}930{,}000 = 98{,}570{,}000 \text{ ft.-lb.}$$
$$M_5 = -11{,}689{,}000 - 2{,}010{,}000 =, \text{ say, } -13{,}700{,}000 \text{ ft.-lb.}$$

Case (*c*).—Moving loads acting.—The uniformly-distributed load of 82 lb. per square foot is equal to 2690 lb. per foot and produces the following bending moments if an additional 15 per cent. is included for impact.

$$M_{10} = 2690 \times 8188 \times 1{\cdot}15 = 25{,}330{,}000 \text{ ft.-lb.}$$

$$M_{5(\text{maximum negative})} = 2690 \times (-3230) \times 1{\cdot}15 = -9{,}991{,}000 \text{ ft.-lb.}$$
$$M_{5(\text{maximum positive})} = 2690 \times 2048 \times 1{\cdot}15 = 6{,}332{,}000 \text{ ft.-lb.}$$

Using the influence-line diagrams it can be shown that the maximum bending moments produced by the test-vehicles, including impact, are $M_{10} = 4{,}960{,}000$ ft.-lb., $M_{5(\text{maximum negative})} = -4{,}895{,}000$ ft.-lb., and $M_{5(\text{maximum positive})} = 2{,}342{,}000$ ft.-lb. Consequently the extreme bending moments on the bridge when loaded most adversely are

$$M_{10} = +98{,}570{,}000 + 25{,}330{,}000 + 4{,}960{,}000 = +128{,}860{,}000 \text{ ft.-lb.}$$
$$M_{5(\alpha)} = -13{,}700{,}000 + 6{,}330{,}000 + 2{,}340{,}000 = -5{,}030{,}000 \text{ ,,}$$
$$M_{5(\beta)} = -13{,}700{,}000 - 9{,}910{,}000 - 4{,}890{,}000 = -28{,}500{,}000 \text{ ,,}$$

Secondary Bending Moments.

As shown in " Prestressed Concrete," by Professor Magnel, the statically-indeterminate reactions on a beam supported at more than two points are altered by prestressing, the amount by which they are altered depending on the position of the cable. The change in the reactions produces secondary bending moments on the beam. The secondary bending moments are by no means negligible.

It is now necessary to compute the secondary bending moment M at point 10. From formula (51) on page 106 of the book mentioned,

$$M_{A1} = -P\frac{\int_0^1 \frac{I}{I} e x_1 . dx_1}{\int_0^1 \frac{I}{I} x_1^2 . dx_1},$$

the terms in which are evaluated in *Table* III. By substitution, $M_{1A} = -5{\cdot}39\,P$. Therefore initially

$$M_{A1} = -5{\cdot}39 \times 12{,}456{,}000 = -67{,}138{,}000 \text{ ft.-lb.},$$

and in course of time

$$M_{A1} = 0{\cdot}85 \times (-67{,}138{,}000) = -57{,}069{,}000 \text{ ft.-lb.}$$

TABLE III.—TERMS FOR CALCULATION OF SECONDARY BENDING MOMENT.

Point	$\frac{e}{I}$	x_l	$\frac{e}{I}x_l$	x_l^2	$\frac{1}{I}$	$\frac{1}{I}x_l^2$
0	0.3820×10^{-2}	0	0	0	3.521×10^{-2}	0
1	0.7700 ,,	0.1	0.0770×10^{-2}	0.01	3.927 ,,	0.0393×10^{-2}
2	1.0750 ,,	0.2	0.2150 ,,	0.04	3.968 ,,	0.1560 ,,
3	1.1800 ,,	0.3	0.3540 ,,	0.09	3.597 ,,	0.3240 ,,
4	1.1750 ,,	0.4	0.4700 ,,	0.16	3.279 ,,	0.5250 ,,
5	1.0870 ,,	0.5	0.5430 ,,	0.25	2.980 ,,	0.7400 ,,
6	0.8260 ,,	0.6	0.4950 ,,	0.36	2.538 ,,	0.9150 ,,
7	0.5150 ,,	0.7	0.3600 ,,	0.49	1.840 ,,	0.9010 ,,
8	0.2770 ,,	0.8	0.1760 ,,	0.64	1.282 ,,	0.8200 ,,
9	0.0057 ,,	0.9	− 0.0042 ,,	0.81	0.490 ,,	0.3970 ,,
10	0.0740 ,,	1.0	− 0.0730 ,,	1.00	0.181 ,,	0.1810 ,,
Totals .			+ 0.02651			+ 0.004927

Summary of the Bending Moments.

A summary of the bending moments to be used for computing the stresses is

(1).—Prestressing just completed.

			Secondary Bending Moments.	
M_{10}	=	+ 84.642 × 10^6 ft.-lb.	M = − 67.138 × 10^6	ft.-lb.
M_5	=	− 11.689 × 10^6 ,,	M = − 33.511 × 10^6	,,

(2).—Construction completed (initially); no live load.

M_{10}	=	+ 98.57 × 10^6 ft.-lb.	M = − 67.138 × 10^6	ft.-lb.
M_5	=	− 13.7 × 10^6 ,,	M = − 33.511 × 10^6	,,

(3).—Construction completed (in course of time); no live load.

M_{10}	=	+ 98.57 × 10^6 ft.-lb.	M = − 57.069 × 10^6	ft.-lb.
M_5	=	− 13.7 × 10^6 ,,	M = − 28.486 × 10^6	,,

(4).—Construction completed (initially); live load acting.

M_{10}	=	+ 128.86 × 10^6 ft.-lb.	M = − 67.138 × 10^6	ft.-lb.
$M_{5(\alpha)}$	=	− 5.03 × 10^6 ,,	M = − 33.511 × 10^6	,,
$M_{5(\beta)}$	=	− 28.5 × 10^6 ,,	M = − 33.511 × 10^6	,,

(5).—Construction completed (in course of time); live load acting.

M_{10}	=	+ 128.86 × 10^6 ft.-lb.	M = − 57.069 × 10^6	ft.-lb.
$M_{5(\alpha)}$	=	− 5.03 × 10^6 ,,	M = − 28.486 × 10^6	,,
$M_{5(\beta)}$	=	− 28.5 × 10^6 ,,	M = − 28.486 × 10^6	,,

Calculation of Stresses.

In computing the stresses the extreme fibre, namely, the highest point of the handrail kerb, will be considered.

Point (10).—A = 157.7 sq. ft.; I = 5612 ft.^{4}; $y_{1(max.)}$ = 10.2 ft.; y_2 = 6.27 ft.; r^2 = 35.9 ft.^{2}; $\frac{r^2}{y_{1(max.)}}$ = 3.52 ft.; $\frac{r^2}{y_2}$ = 5.74 ft.; $\frac{I}{y_1}$ = 549 ft.^{3}; and $\frac{I}{y_2}$ = 893 ft.^{3}

The stresses produced by the initial secondary bending moment are

$$c_t = \frac{67{\cdot}138 \times 10^6}{549 \times 144} = +\ 847 \text{ lb. per square inch;}$$

$$c_b = \frac{67{\cdot}138 \times 10^6}{893 \times 144} = -\ 522 \text{ lb. per square inch.}$$

In course of time, these stresses become $c_t = +\ 720$ lb. per square inch and $c_b = -\ 444$ lb. per square inch.

The various bending moments due to the loads which have to be considered produce the following stresses in lb. per square inch.

Bending moments (ft.-lb.).	c_t	c_b
$+\ 84{\cdot}642 \times 10^6$	$-\ 1069$	$+\ 658$
$+\ 98{\cdot}57\ \times 10^6$	$-\ 1248$	$+\ 766$
$+\ 128{\cdot}86\ \times 10^6$	$-\ 1632$	$+\ 1002$

The prestress alone produces initially the following stresses:

$$c_t = \frac{12{\cdot}456 \times 10^6}{157{\cdot}7 \times 144}\left(1 + \frac{4{\cdot}30}{3{\cdot}52}\right) = 1236 \text{ lb. per square inch}$$

$$c_b = \frac{12{\cdot}456 \times 10^6}{157{\cdot}7 \times 144}\left(1 - \frac{4{\cdot}30}{5{\cdot}74}\right) = 135 \quad \text{,,} \quad \text{,,}$$

and in course of time these stresses become $c_t = 1051$ lb. per square inch, and $c_b = 115$ lb. per square inch, both stresses being compressive.

The resulting stresses for the five cases mentioned are:

Case (1). $c_t = -\ 1069 + 847 + 1236 = +\ 1014$ lb. per square inch.
$c_b = +\ 658 - 522 + 125 = +\ 271$,, ,, ,,

Case (2). $c_t = -\ 1248 + 847 + 1236 = +\ 835$,, ,, ,,
$c_b = +\ 766 - 522 + 135 = +\ 379$,, ,, ,,

Case (3). $c_t = -\ 1248 + 720 + 1051 = +\ 523$,, ,, ,,
$c_b = +\ 766 - 444 + 115 = +\ 437$,, ,, ,,

Case (4). $c_t = -\ 1632 + 847 + 1236 = +\ 451$,, ,, ,,
$c_b = +\ 1002 - 522 + 135 = +\ 615$,, ,, ,,

Case (5). $c_t = -\ 1632 + 720 + 1051 = +\ 139$,, ,, ,,
$c_b = +\ 1002 - 444 + 115 = +\ 673$,, ,, ,,

Point (5).—$A = 60{\cdot}5$ sq. ft.; $I = 335$ ft.^{4}; $y_1 = 2{\cdot}76$ ft.; $y_2 = 4{\cdot}49$ ft.; $r^2 = 5{\cdot}54$ ft.^{2}; $\frac{r^2}{y_1} = 2{\cdot}013$ ft.; $\frac{r^2}{y_2} = 1{\cdot}253$ ft.; $\frac{I}{y_1} = 121{\cdot}4$ ft.^{3}; and $\frac{I}{y_2} = 74{\cdot}5$ ft.^{3}

The stresses produced by the initial secondary bending moment are

$$c_t = \frac{33{\cdot}511 \times 10^6}{121{\cdot}4 \times 144} = +\ 1919 \text{ lb. per square inch;}$$

$$c_b = \frac{33{\cdot}511 \times 10^6}{74{\cdot}5 \times 144} = -\ 3128 \text{ lb. per square inch,}$$

which become, in course of time, $c_t = 1631$ lb. per square inch and $c_b = -\ 2659$ lb. per square inch.

The various bending moments, due to the loads, which have to be considered produce the following stresses in lb. per square inch.

		Bending moments (ft.-lb.)	c_t	c_b
Case (1).		$-11{\cdot}69 \times 10^6$	+ 668	− 1089
Cases (2) and (3).		$-13{\cdot}7 \times 10^6$	+ 782	− 1277
Cases (3) and (4).	(α)	$-5{\cdot}03 \times 10^6$	+ 287	− 469
	(β)	$-28{\cdot}5 \times 10^6$	+ 1635	− 2659

The prestress alone produces initially the following stresses:

$$c_t = \frac{12{\cdot}456 \times 10^6}{60{\cdot}5 \times 144}\left(1 - \frac{3{\cdot}64}{2{\cdot}013}\right) = -1159 \text{ lb. per square inch}$$

$$c_b = \frac{12{\cdot}456 \times 10^6}{60{\cdot}5 \times 144}\left(1 + \frac{3{\cdot}64}{1{\cdot}233}\right) = +5617 \quad \text{,,} \quad \text{,,} \quad \text{,,}$$

which in course of time become $c_t = -984$ lb. per square inch and $c_b = +4778$ lb. per square inch.

The resulting stresses for the five cases are:

Case 1.	$c_t = +668 + 1919 - 1159 = +1428$	lb. per square inch.
(initially)	$c_b = -1089 - 3128 + 5617 = +1400$	,, ,,
Case 2.	$c_t = +782 + 1919 - 1159 = +1542$	,, ,,
(initially)	$c_b = -1277 - 3128 + 5617 = +1212$	,, ,,
Case 3.	$c_t = +782 + 1635 - 984 = +1433$	,, ,,
(in course of time)	$c_b = -1277 - 2659 + 4778 = +842$	,, ,,
Case 4(α).	$c_t = +287 + 1919 - 1159 = +1047$	,, ,,
(initially)	$c_b = -469 - 3128 + 5617 = +2020$	,, ,,
Case 4(β).	$c_t = +1635 + 1919 - 1159 = +2395$	,, ,,
(initially)	$c_b = -2659 - 3128 + 5617 = -170$	,, ,,
Case 5(α).	$c_t = +287 + 1635 - 984 = +938$	,, ,,
(in course of time)	$c_b = -469 - 2659 + 4778 = +1650$	,, ,,
Case 5(β).	$c_t = +1635 + 1635 - 984 = +2285$	,, ,,
(in course of time)	$c_b = -2659 - 2659 + 4778 = -540$	,, ,,

From the foregoing it is seen that the extreme stresses are:

At A_1 (support): + 139 lb. per square inch and + 1014 lb. per square inch.
At B_1 (midspan): − 540 lb. per square inch and + 2395 lb. per square inch.

These stresses are permissible if the crushing strength of the concrete cubes is not less than $4 \times 2395 = 9580$ lb. per square inch and if some ordinary steel reinforcement is provided at B_1 in order to resist the small tensile force. It must not be forgotten, however, that the maximum compressive stress occurs only at the top of the small handrail kerb. If the state of stress at this point were such that the elastic modulus of the concrete greatly decreased, the slab of the roadway would have to resist greater stresses, but at the extreme fibre of this slab the stress is not so great as at the kerb and, for case 4(β), it is 2159 lb. per square inch. This stress does not exceed the permissible compressive stress of 2160 lb. per square inch, and requires concrete having a cube strength of $4 \times 2159 = 8636$ lb. per square inch.

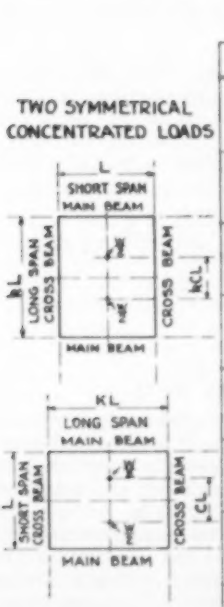
Concentrated Loads on Slabs Spanning in Two Directions.

By L. CHALLEN, A.M.Inst.E.(Australia).

THE accompanying table has been prepared to facilitate the design of slabs spanning in two directions and carrying one or two loads symmetrically disposed. The basis of the calculation is a combination of the methods of Professor Löser and Dr. Marcus as given by Mr. N. A. Dews in this journal for September, 1946, and as modified by the writer in this journal for October, 1947. The slab is assumed to be rigidly fixed when calculating the bending moments over the main and cross beams and fixed to the extent of half complete fixity when calculating the moments at mid-span.

The method of using the table is to determine C (the load-spacing factor) and k, the ratio of the distances between the main beams and the cross beams, or K, which equals $\frac{1}{k}$. The table is limited to values of kC not greater than unity. For k or K (whichever exceeds unity) and C find the corresponding

TABLE I.—COEFFICIENTS FOR SLABS SPANNING IN TWO DIRECTIONS AND SUPPORTING CONCENTRATED LOADS.



$C = \frac{\text{DISTANCE BETWEEN LOADS}}{\text{DISTANCE BETWEEN MAIN BEAMS}}$

	C	0	0·1	0·2	0·3	0·4	0·5	0·6	0·7	0·8	0·9	
TWO SYMMETRICAL CONCENTRATED LOADS	k = 1·40	0·144	0·143	0·143	0·141	0·139	0·137	0·132	0·126			a
		0·110	0·075	0·039	0·007	----	----	----	----			b
		0·116	0·116	0·115	0·113	0·109	0·105	0·098	0·088			c
		0·056	0·056	0·055	0·056	0·056	0·056	0·056	0·053			d
	k = 1·30	0·143	0·142	0·142	0·140	0·138	0·134	0·130	0·123			a
		0·113	0·080	0·049	0·021	----	----	----	----			b
		0·115	0·115	0·115	0·110	0·106	0·101	0·095	0·082			c
		0·063	0·063	0·064	0·064	0·064	0·064	0·062	0·059			d
	k = 1·20	0·141	0·141	0·140	0·138	0·133	0·132	0·126	0·120	0·110		a
		0·118	0·086	0·061	0·035	0·012	----	----	----	----		b
		0·111	0·111	0·109	0·106	0·101	0·095	0·086	0·074	0·057		c
		0·073	0·073	0·074	0·074	0·073	0·072	0·069	0·063	0·053		d
	k = 1·10	0·139	0·139	0·137	0·135	0·132	0·128	0·123	0·116	0·107	0·095	a
		0·126	0·099	0·074	0·052	0·031	0·013	----	----	----	----	b
		0·104	0·104	0·101	0·098	0·093	0·086	0·076	0·064	0·048	0·027	c
		0·084	0·084	0·084	0·083	0·082	0·079	0·075	0·067	0·054	0·035	d
	k = K = 1·00	0·137	0·137	0·136	0·133	0·130	0·126	0·121	0·114	0·106	0·096	a
		0·137	0·113	0·091	0·071	0·053	0·037	0·023	0·012	0·004	----	b
		0·096	0·093	0·091	0·087	0·081	0·076	0·065	0·053	0·038	0·021	c
		0·096	0·096	0·093	0·092	0·089	0·085	0·079	0·069	0·054	0·032	d
	K = 1·10	0·126	0·125	0·124	0·122	0·120	0·116	0·112	0·106	0·100	0·093	a
		0·139	0·116	0·095	0·076	0·059	0·044	0·030	0·019	0·010	0·004	b
		0·084	0·083	0·080	0·076	0·071	0·063	0·054	0·044	0·031	0·016	c
		0·104	0·104	0·103	0·101	0·097	0·091	0·085	0·071	0·054	0·031	d
	K = 1·20	0·118	0·118	0·117	0·115	0·113	0·110	0·106	0·102	0·098	0·092	a
		0·141	0·119	0·099	0·081	0·064	0·049	0·035	0·025	0·014	0·006	b
		0·073	0·072	0·070	0·066	0·061	0·054	0·045	0·036	0·025	0·013	c
		0·111	0·111	0·109	0·106	0·101	0·094	0·084	0·071	0·053	0·030	d
	K = 1·30	0·113	0·113	0·112	0·111	0·109	0·106	0·104	0·101	0·097	0·093	a
		0·143	0·121	0·102	0·084	0·067	0·052	0·038	0·026	0·016	0·007	b
		0·063	0·062	0·060	0·056	0·051	0·045	0·038	0·029	0·020	0·010	c
		0·115	0·114	0·112	0·109	0·105	0·095	0·084	0·070	0·051	0·028	d
	K = 1·40	0·110	0·110	0·109	0·106	0·107	0·105	0·103	0·100	0·098	0·095	a
		0·144	0·123	0·104	0·086	0·069	0·054	0·041	0·028	0·018	0·008	b
		0·054	0·053	0·051	0·048	0·044	0·038	0·032	0·026	0·016	0·008	c
		0·116	0·116	0·115	0·109	0·103	0·094	0·083	0·068	0·049	0·027	d

values of the coefficients *a*, *b*, *c*, and *d* from the table. Multiply each coefficient by *W* to give the bending moment on unit width of slab if *W* is the sum of the two concentrated loads. The coefficients are dimensionless; therefore, if *W* is in pounds, the products are in ft.-lb. per foot width of slab. The bending moments at the critical sections are as follows.

(i) *Wa* is the bending moment at mid-span and determines the reinforcement in the bottom of the slab parallel to the main beams.

(ii) *Wb* is as (i) but for the reinforcement parallel to the cross beams.

(iii) *Wc* is the bending moment over the cross beams and determines the reinforcement in the top of the slab parallel to the main beams.

(iv) *Wd* is the bending moment over the main beams and determines the reinforcement in the top of the slab parallel to the cross beams.

Example.

Consider a panel the span of which between the main beams is 10 ft. and between the cross beams is 12 ft., that is $L = 10$ ft., $KL = 12$ ft., and $K = 1{\cdot}2$. A single wheel imposes a load of 20 tons ($W = 44{,}800$ lb.) on the slab. The contact area on the wearing surface (assumed to be 3 in. thick) is assumed to be 3 in. in the direction of KL and 24 in. in the direction of L. By dispersion at 45 deg. through the wearing surface, the loaded area at the top of the slab is 9 in. by 30 in. The single load on this area can be replaced by two "point" loads each of 22,400 lb. and spaced so that $CL = 0{\cdot}5 \times 30$ in. $= 1{\cdot}25$ ft. Therefore $C = 1{\cdot}25 \div 10 = 0{\cdot}125$.

From the table, for $K = 1{\cdot}2$ and $C = 0{\cdot}125$, by interpolation, $a = 0{\cdot}118$, $b = 0{\cdot}116$, $c = 0{\cdot}072$, and $d = 0{\cdot}111$. The bending moments (in ft.-lb. per foot width) due to the wheel load only are:

(i)	At mid-span, parallel to main beams:	$44{,}800 \times 0{\cdot}118 = 5306$.
(ii)	,, ,, ,, ,, cross ,,	$44{,}800 \times 0{\cdot}116 = 5197$.
(iii)	Over the cross beams:	$44{,}800 \times 0{\cdot}072 = 3236$.
(iv)	,, ,, main ,,	$44{,}800 \times 0{\cdot}111 = 4973$.

An Early Reinforced Concrete Jetty.

We have received the following from Mr. R. B. Kirwan, a Director of Messrs. Samuel Williams & Sons, Ltd., of Dagenham Dock, Essex.

"After reading the articles in the December, 1949, and February, 1950, numbers of 'Concrete and Constructional Engineering' on the early history of reinforced concrete, I was interested to come across a reference in your March, 1911, number (page 234) to early examples of reinforced concrete in this country. One was a reinforced concrete beer cooler apparently constructed between 1850 and 1860, and the other a flat roof of coke-breeze concrete 4 in. thick reinforced with flat and round steel bars constructed in 1876.

"It may be of interest to you to know that we still have in use at Dagenham our No. 4 Jetty, which was constructed between 1901 and 1902 and was the first reinforced concrete jetty to be built on the Thames. It was designed by the late L. G. Mouchel, and constructed by Messrs. J. C. Lang & Jones, Ltd."

Book Reviews.

"Calculation, Design and Testing of Reinforced Concrete." By K. L. Rao, M.Sc., Ph.D. (London: Sir Isaac Pitman & Sons, Ltd. 1950. Price 40s.)

THIS book is good, and fulfils a need. It contains nearly 400 pages and gives many well-chosen and fully worked out numerical examples extracted from questions set in recent years for B.Sc. examinations at the Universities of London and Birmingham and also at qualifying examinations for membership of the Institution of Civil Engineers and the Institution of Structural Engineers. The book deals with the fundamental principles of design in reinforced concrete and tests of materials, and gives examples of structures. It also contains useful appendixes. The examples show how thoroughly reinforced concrete design is taught in technical colleges.

While there is so much to be said in favour of this book and its aims, it is unfortunate that it is already somewhat out of date. It refers throughout to the new Code of Practice, but this is the D.S.I.R. Code issued in the year 1934 and not the British Standard Code (CP. 114) of 1948 as one would expect in a book published in 1950. Some of the diagrams are not clear, for example, the drawing on p. 43, although correctly dimensioned, is entirely out of scale and therefore fails to train the eye of a student to the appearance of a standard hook, and again, on p. 100, the tee-beam bars, shown bent to resist shearing forces, appear to be more like floor-slab bars. The chapter on shear is hard to follow, and lacks a few well-drawn examples of shear reinforcement so necessary to illustrate the text. The same may be said of the example of the tee-beam on p. 102. On p. 228 there is a lack of clarity in the assumptions made in getting useful but approximate results, and a confusion between w as the load per foot span and W the load per span, where W would normally be equal to wl; also, at the bottom of this page, there appears to be a misprint in signs. This lack of clarity might puzzle the novice for whom the book is intended. On p. 271 the description of an in-situ pile presents difficulty.

It is hoped that the book may be revised at an early date and provided with a few more illustrations. It would then be of greater value to the student entering for examinations and to engineers who wish to refer to examples derived from first principles. It is hoped also that the price may be reduced.—R. P. M.

"Soil Mechanics in Road Construction." By C. F. Armstrong. 1950. (London: Edward Arnold & Co. Price 30s.)

SOIL mechanics "is a young and rapidly developing science wherein a number of theories and empirical formulæ have been put forward for the solution of problems of stability and deformation of soils. Their proper interpretation, however, is in many cases not fully established and their application still remains to be systematically checked by controlled observations during works of construction. Soil mechanics . . . must never be looked upon as a substitute for civil engineering experience." These sentences are from the author's preface, and should be borne in mind particularly by young engineers who may think that soil mechanics is a substitute for experience. The book is an excellent survey of the principles underlying the theory and the present practice of the subject.

"Der Stahlbetonbau." By R. Saliger. Seventh edition. (Vienna: Franz Deuticke. 1949. Price 50 D.M.)

THE principal additions to the present edition of this standard text-book in the German language are greater consideration of the influence on the strength of concrete of the grading and surface-area of gravel aggregates, the conditions required for obtaining dense concrete, creep of concrete, the properties of steel having a high yield-point stress, and the pre-tensioning of steel. The author's new theory of the formation of cracks in beams is described, and further consideration is given to plastic flow based on recent research. A method of design is described based on a plastic theory.

Numerous design tables, which are independent of the dimensional system used, are given. Improvements have been made in the chapter on bending combined with axial forces, and greater use is made of the conception of a lattice to demonstrate the action of reinforcement to resist shearing forces. The theoretical consideration of flat-slabs and slabs supported on all edges has been considerably extended. Simple solutions are presented for dealing with sway in

complicated frames. The position of expansion joints and hinges is discussed in relation to the entire structure.

There are more than 600 pages, 700 illustrations, 140 design tables and diagrams, and extensive bibliographies. Although the main purpose is to provide a practical text-book for students and designers, emphasis is laid on the scientific aspect of structural engineering in order to foster in the reader a critical appreciation of the subject, without which the author considers that engineers cannot be creative but remain slaves to mechanical calculation. Dr. Saliger is a purist in regard to the German language and avoids almost fanatically words of foreign origin. For example, "plastic theory" is not given as "plasticitäts Theorie" but as "bildsamer Bereich." The reader may therefore experience some difficulty even if he is familiar with ordinary technical German. This is only a small matter, however, and does not detract from the qualities of a book which embodies the long experience of an authority on reinforced concrete.—H.V.C.

"A.S.T.M. Standards on Industrial Water." (Philadelphia: American Society for Testing Materials. 1949. Price $1.75.)

IN this booklet of 142 pages are given twenty-six American standard methods of sampling, analysing, and testing water. The methods, some of which are new, some are revisions or repetitions of previous standard methods, and some are tentative, apply particularly to water for the generation of steam, or industrial processes, or cooling purposes. Some methods, such as those for the determination of acidity and alkalinity, the content of carbon-dioxide, sulphate, and chloride, and the presence of suspended or dissolved solids, should be of value when testing water for making concrete.

"Metric Conversion Tables." (London: Frederick Warne & Co., Ltd. 1950. Price 15s.)

IN about one hundred pages of tables and other data are given factors for the conversion of British and American measures of weight, length, speed, area, volume, and capacity to the corresponding measures in the metric system. Other tables give the conversion of British money into decimals of the pound sterling, thermometer scales, and pressures. The tables of pressures give, among other data, pounds per square inch in terms of kilogrammes per square centimetre, but such tables are not so comprehensive as others published elsewhere. It is pointed out that the exact ratios between systems of measurement developed independently are difficult to establish. In this respect the American and British factors for conversion to the metric system differ. For example, the legal equivalent of 1 metre in America is 39·370000 in., and in Britain, 1·0936143 yards, that is 39·370115 in. to six places of decimals. In general, conversion factors for length, weight, and capacity can be relied on only to 1 in 1, 5, and ½ million respectively, and American conversions of lengths differ by 3 in a million from the British figures.

"Einflussflächen für Trägerroste." By H. Homberg. Volume I. (Westfalen: Published by the Author. 1949. No price stated.)

THIS book of 59 pages in the German language describes a method of calculating the forces and bending moments on grids of beams in which any number of parallel main longitudinal beams span freely over one opening only, and in which there are one to nine or an infinite number of transverse beams. Since such structures are highly statically indeterminate, the author provides the data for numerous influence diagrams from which solutions can be quickly and accurately obtained. The method, which is approved by the German transport authority, is mainly applicable to bridges the decks of which are grids of steel beams.

"Holz-Nagelbau." By F. Fonrobert and W. Stoy. Sixth edition. (Berlin: Wilhelm Ernst & Sohn. 1949. Price 3.60 DM.)

THIS is a practical booklet describing nailed timber joints for temporary wooden structures. Nailed timber construction is sometimes used in the centering for arch bridges, and two examples are illustrated.

Books Received.

"R.C.C. Designing Made Easy." By R. S. Deshpande. 1950. (Poona: Union Book Corporation. Price 10s.)

"Clay Building Bricks of the United Kingdom." National Brick Advisory Council. Paper No. 5. 169 pages. 10 in. by 6 in. (London: His Majesty's Stationery Office. 1950. Price 7s. 6d.)

"Resistance Welding in Mass Production." By A. J. Hipperson and T. Watson. (London: Iliffe & Sons, Ltd. 1950. Price 21s.)

Precast Concrete Signal Gantries.

For the support of coloured-light signals on some lines in the London suburban area, the Southern Region of British Railways has recently erected precast reinforced concrete cantilevered gantries (*Fig.* 1). The dimensions of a typical gantry and the arrangement of the reinforcement therein are shown in *Fig.* 2.

Fig. 1.

In addition to ability to withstand the ordinary dead and live loads without appreciable deflection, features of the design are low maintenance and considerable rigidity against twisting in a horizontal plane due to wind pressure on the structure and signals. Factors resulting in reduction of maintenance costs include the use of a concrete of great strength, comparatively low working stresses (thereby reducing the risk of cracks), and non-corrodible metal attachments such as steel hand-rails and ladders sprayed with aluminium wire, and heavily-galvanised tubes, plates, etc.

The vertical live load on the gantry is assumed to be equivalent to 20 lb. per square foot, which allows for men and snow on the horizontal arm. The weight of a four-aspect signal is about 2 cwt., and 4 cwt. if combined with a junction indicator. The pressure of the wind is calculated in accordance with the British Standard Code, Chapter V, Loading, assuming the effective unsheltered height to be 20 ft. and the velocity of the wind to be 75 miles per hour. Allowing for an impact factor of 150 per cent., the assumed pressure of the wind is 35 lb. per square foot. The torsional rigidity in a horizontal plane is specified to be such that the angular deformation must not exceed 1 deg., otherwise the beam of light from the signal is thrown out of alignment and cannot be easily discerned from some distance away along the track. The actual angular deformation is only about half the permissible amount. The twisting moment in the post is resisted entirely by the double (and contrariwise) helical reinforcement shown in *Fig.* 2. The torsional reinforcement comprises bars of ¼ in. diameter and is at 45 deg. Any diametral plane intersects four bars. The calculated tensile stress in the bars is about 7000 lb. per square inch.

The permissible stresses are a compressive stress of 1000 lb. per square inch in the concrete and a tensile stress of 25,000 lb. per square inch in the steel, but in all parts of the structure the calculated stresses under the most adverse conditions are much less than those permitted. The modular ratio used in the calculations is 15. The main reinforcement is hot-rolled high-tensile plain round steel bars. Other reinforcement is mild steel. All reinforcement complies with British Standard No. 785.

Manufacture and Erection.

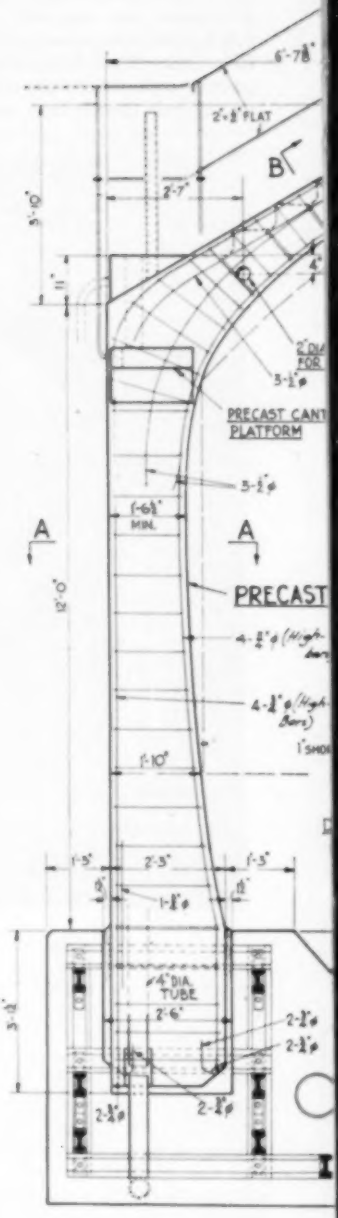
The gantries were manufactured at the Southern Region concrete depot at Exmouth Junction. The concrete is mixed in the proportions of about 4 cu. ft. of ½-in. graded granite screenings, 2 cu. ft. of sand, and 1 cwt. of rapid-hardening

Portland cement. The water-content for 1 cwt. of cement is 5 gallons, but this is reduced to allow for the measured amount of moisture in the aggregates. The concrete is consolidated by immersion vibrators, and the average strengths of 6-in. cubes are 5200 lb. per square inch at seven days and 6750 lb. per square inch at 28 days. The moulds in which the gantries are cast in one piece, except for a small projecting ladder-platform, are of wood and are lined on the inner faces with metal.

The helical binding is made by first winding on the steel drum of a winch the requisite number of complete turns of ½-in. bar to form a close coil of about 19½ in. diameter. This size was determined by test so that, when stretched, the coil is of the required diameter, about 14½ in., of the completed helical. The close coil is taken off the drum and one end is fixed in a pipe-vice. The other end is attached to a chain on a winch which pulls the coil open until it is of the required length, pitch and slope. Two coils are then slipped over a long wooden core so that one coil is inside the other and the direction of the twist of the inner coil is opposite to that of the outer. Where the bars of the two coils pass each other a spot-welded connection is made and a rigid cage of reinforcement is formed.

When the casting is removed from the mould it is placed over a trestle so that the face that is downwards during casting is exposed and can be rubbed to a fair finish. All arrises are rounded to about ⅜-in. radius. The gantry is raised off the trestle by a 5-tons steam crane and placed on a special railway wagon on which it is transferred to the site where it is to be erected. The foundations comprise a cast-in-situ block of 1 : 2 : 4 concrete, with 1½-in. aggregate, reinforced with rails. The post of the gantry is lowered so that the end is inserted into a socket in the top of the block. The 1½-in. space between the sides of the post and socket are filled with sand except for a depth of about 6 in. at the top which is filled with fine concrete. The size of the foundation varies to suit the site. The method of fixing the post enables the gantry to be dismantled readily should it be necessary to move the track.

The foregoing particulars and illustrations of the gantries, which are being



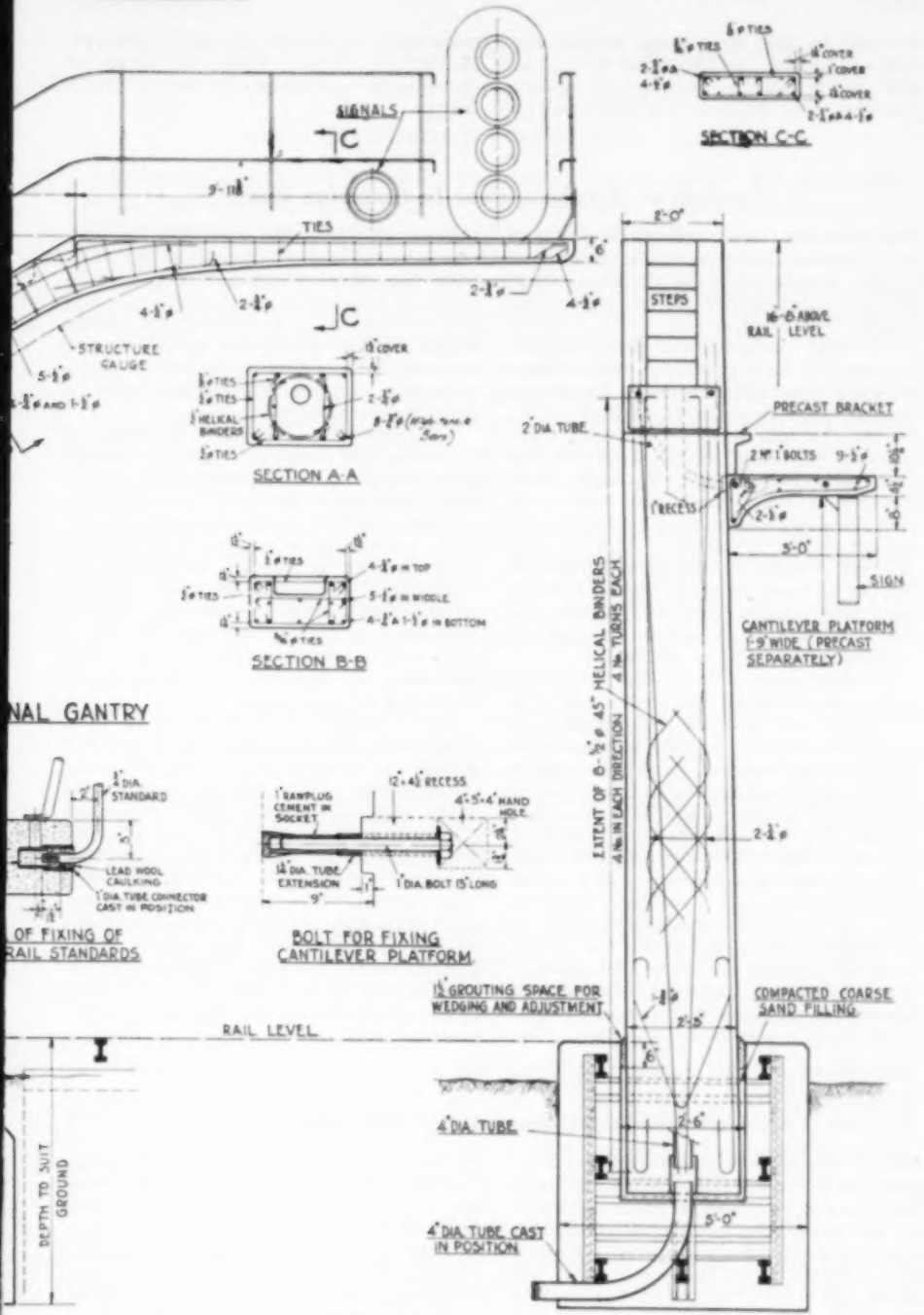


Fig. 2.—Precast Concrete Signal Gantry.

erected on the line from Bricklayers' Arms Junction to Norwood Junction, and at Poupart's Junction, Clapham Junction, and Balham, are published by permission of Mr. V. A. M. Robertson, M.Inst.C.E., Chief Civil Engineer of the Southern Region of British Railways.

Continuous Reinforcement in Concrete Roads.

THE following is a summary of a report of an investigation by the U.S. Bureau of Public Roads and the Indiana State Highway Commission, given in "Public Roads" for April, 1950.

Concrete roads without closely-spaced transverse joints generally crack transversely at frequent intervals. The cracks tend to open appreciably in time unless the slab is reinforced. For some years highway engineers have considered the practicability of concrete roads constructed without transverse joints and with sufficient longitudinal reinforcement to hold the cracks closed. In the autumn of 1938 several continuously-reinforced lengths of road, from 20 ft. to 1310 ft. long, were constructed in Indiana, to enable the effects of various amounts of longitudinal steel to be studied.

The behaviour of the roads during the first ten years shows that continuous reinforcement will prevent the opening of transverse cracks. In long sections with the greatest amount of reinforcement many fine superficial cracks developed in the central region. The cracks have not opened and have ravelled only slightly due to traffic and exposure, but have required no maintenance. The road remains strong and durable. The presence of even the greatest amount of longitudinal bar reinforcement has apparently not affected adversely the concrete, which appears to be sound throughout, there being no spalling and an absence of longitudinal cracks above the bars. The manner in which the steel has held all cracks closed, especially those in the parts with the greatest amount of reinforcement, is believed to be conducive to distributed interfacial pressure at the cracks which tends to reduce the damage to the concrete from concentrations of pressure such as sometimes develop at cracks in plain concrete slabs.

"Pumping" has occurred at many of the transverse joints but, with two exceptions, has not been observed at any of the many transverse cracks, indicating that a concrete road without transverse joints and containing adequate longitudinal reinforcement is not so susceptible to "pumping" as roads of other designs. In spite of the many transverse cracks that occur in the long sections, the riding quality of the roads remains excellent and the road is therefore protected from damaging impact forces.

The experimental roads are 20 ft. wide with a central longitudinal joint and the slab is 7 in. thick at the middle and 9 in. at the edges. The longitudinal reinforcement comprises deformed round bars from ½ in. to 1 in. diameter or a mesh of cold-drawn wire from 0·2 in. to 0·4 in. diameter. The longitudinal bars or wires are at 6-in. centres. The transverse reinforcement is roughly proportional to the longitudinal reinforcement and varies from ½-in. deformed bars at 12-in. centres to ½-in. bars at 24-in. centres, or 0·2-in. wire at 12-in. centres to 0·24-in. wire at 12-in. centres.

Materials transported by Helicopter.

ABOUT 170 tons of material and equipment for the construction of a small rock-filled dam at an otherwise inaccessible site in the mountains of British Columbia were transported by helicopter. The rock was obtained from the excavation, but about 100 tons of aggregate, 24 tons of cement, 3 tons of steel, and 10 tons of timber were transported by helicopter. The maximum load on each journey was 400 lb. if evenly distributed. The distance of the flight was about five miles and the landing space was 15 ft. square. Weather unfavourable for flying was one of the greatest difficulties, but the experiment was otherwise successful.

Construction with Moving Forms—IV.*

By L. E. HUNTER, M.Sc., A.M.Inst.C.E.

Procedure of Construction.

EXPERIENCED supervision is highly important to avoid grave mistakes, which may be very costly to rectify. Every endeavour must be made to ensure that nothing shall go wrong, and it is very unusual if difficulties do not occur while the work is in progress. The writer knows of an occasion when the contractor's agent was on the site for thirty-six hours without a break due to the work being extremely difficult at this stage. The actual moving of the forms is the result of a carefully-thought-out programme. The preparations may proceed over a period of months, and the stages are as follows:

(1) The foundations are concreted while the forms are being made.

(2) The lower part of the superstructure and walls are concreted within fixed shutters, and the moving forms are fixed in place as this work proceeds.

(3) The moving forms, the access scaffold, staircases, plant, and materials are placed in position.

(4) The reinforcement bars and jack-rods are placed in position.

(5) The forms are cleaned, concrete is deposited in the forms throughout, and the upward movement of the forms is started.

(6) The moving-form construction is completed, the roof or other covering slab is concreted, and the forms are dismantled.

Preliminaries.

While the foundations are being concreted, and not later, the preparatory work on the forms proceeds. A separate shed should be used for the assembly of parts of the forms, and full-scale templates drawn on the floor in order to obtain the necessary accuracy. The templates allow the fitting of the four sides of the shutters in the case of rectangular or square bins, and the whole circumference in the case of circular bins. When the forms are correctly made and fitted together, it is essential to mark them so that the parts for each bin can be erected in their proper order.

At the same time the supply of the reinforcement and concrete materials must be attended to, and the mixers or batching plant overhauled and placed in position. The aggregates are not required on the site at this stage, since they require a large storage area and supplies should not arrive until two to three weeks before moving commences.

The erection of the outside staircase should be begun at an early stage, and it can be built to the full height of the structure beforehand. At the same time provision must be made for hoisting the reinforcement. The deck is omitted from one of the bins and a hoist placed in this position. The positions of the hoists and their number depend on the size of the structure.

Construction of Walls.

The first step in the construction is the fixing of the reinforcement for the first 4 ft. to 5 ft. of the height of the wall. The forms are then placed in position

* Continued from March, April, and May.

and the yokes erected. Next the jack-rods are fitted in the yokes. As the construction of the forms proceeds, the decks are boarded over to form a working platform and the racks for reinforcement are erected on top of the yokes. When the forms are in place, and the whole of the plant, concreting materials, reinforcement, and equipment is placed in position, the work of moving can commence. The men have to be allocated to two shifts, and extra men will be necessary. The supervision must be very strict, and work on the deck kept under constant surveillance. It is common to start the moving-form construction at night. After the forms have been thoroughly cleaned out, the carpenters and the steel-fixers make any adjustments that may be necessary. It is essential to prepare carefully the concrete on which the walls are to be constructed by hacking the old concrete and hosing it generously with water. A thin layer of liquid cement grout must be applied before any new concrete is placed.

The concreting is then begun and the full depth of the forms is concreted all around the building before the shutters are gently eased in an upward direction. The first easing of the forms requires considerable caution. At no stage is the strain on the jack-rods more severe, and this part of the work must be undertaken with only $\frac{1}{8}$-in. vertical lift throughout the work at long intervals. At the end of the first shift, however, the forms should have been raised at least 3 in., depending on the temperature. In the second shift the rate of jacking is gradually increased from 3 in. per hour to 6 in. as a uniform rate. At this stage difficulties may be encountered in spite of the care given to the problem. For example, the supply of reinforcement to the platform or of cement to the mixers may not be sufficient, a mixer or a hoist may break down, or the reinforcement may not be placed fast enough. These are all possible sources of trouble throughout the work.

As already stated, moving-form construction, more than any other method of construction, depends for its success on the uniform and consistent supply of materials from the mixing plant to the deck. Should prolonged stoppage of the concreting occur at any stage in the work the result will always be serious and restarting will be accompanied by binding of the forms to the hardened concrete and often by the lifting of the top layers of concrete with the forms. Such a stoppage may be very expensive. For example, the writer had experience of moving-form construction which, because of lighting restrictions during the war, was carried out in twelve-hour daylight shifts. The weather was very hot, and the concrete lifted when moving started again. Most of the walls were affected in this manner and it took four days to free the forms from the raised top layer of concrete. This delay probably cost more than a thousand pounds.

Throughout the work the engineer in charge must be on the site and, with the assistance of the foreman, must constantly go around the work to ensure that it is going well. Not only must he ensure that the work at the top of the forms is in order, but he must make frequent inspections of the outside scaffold to determine the state of the concrete as it leaves the forms. The internal walls must also be inspected at intervals. The concrete when it leaves the forms must be relatively hard without any bulging below the forms. Should such bulging occur it generally means that the jacking is too fast, and the rate of progress must be reduced. Frequently this occurs in the early morning when the temperature is lower, but in the afternoon the speed of the work can often be increased.

The condition of the outer walls indicates the correctness of the rate of progress and of the water-cement ratio with far greater accuracy than do the interior walls because the outer walls are more affected by changes of temperature. The temperature in the internal bins is always high whatever the outside temperature.

The common requirements of the consistency of concrete for wall construction do not apply in the case of moving-form construction. The controlling factor is the amount of reinforcement, and when this is complicated a workable concrete, irrespective of its consistency, is advocated. The time available for the punning of the concrete is limited, and if the mixture is stiff and difficult to place around the bars air pockets will be formed in the wall and " lifting " of the concrete with the forms is likely to occur.

The movement of the forms must not stop during a shift, and between shifts (there is usually an interval of half an hour) the forms should be gradually eased a fraction of an inch at a time to prevent the concrete from sticking to them. The average rate of progress depends largely on the weather as low temperatures, rain, and wind retard the work. If the work is undertaken in warm weather, the average rate of moving might exceed 8 in. per hour. In winter it is frequently difficult to achieve more than 6 in. per hour. Winter work also requires safeguards against frost, such as heating the aggregate and the provision of frost curtains and braziers on the deck, but winter work is rarely as satisfactory as that done in warmer weather.

To aid the fixing of the reinforcement the systematic marking on the vertical bars of the correct positions of the horizontal bars is necessary. It is important to keep a close check on the point where the spacing changes and, to ensure that this occurs at the correct height, the provision of a steel tape fastened to a point at the bottom of the bins is desirable ; the tape is unrolled as the work proceeds, and as the level of the bottom of the tape is known the height of the forms can be read at once.

Levelling the Forms.

A very important operation which should be undertaken by the engineer in charge with the help of one or two men is that of levelling the forms. It is necessary to do this before the moving is started, and thereafter it must be done at regular intervals throughout each shift or preferably in the periods between the shifts. For this purpose a dumpy-level is set up at a convenient place on the deck and the legs of the tripod firmly fixed. Two of the three levelling-screws are tightened so that vertical movement of the collimation line is achieved by the third screw. When levelling has to be undertaken, a staff-man goes around the walls and sets up the staff at each corner. The corners which are shown to be low are brought up to the level of the high points by jacking, after which the level is checked and deviations corrected. It is always preferable to raise the low points rather than to lower the high points.

To enable the level of the forms and deck to be checked at any time without using an instrument, each yoke is fitted with a vertical 12-in. scale (*Fig.* 27) marked in inches and fixed near the jack-rod, which is marked in feet before concreting begins because at this time the forms are known to be level. As jacking proceeds, the marks on the jack-rods are continued upwards. At constant heights above the deck are placed small cross-trees at about eye level. By boning

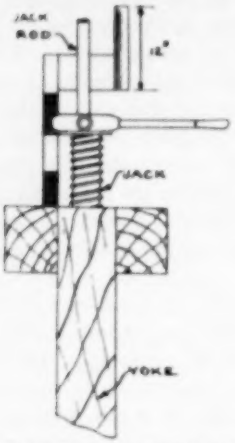


Fig. 27.—Scale Fixed to Yoke.

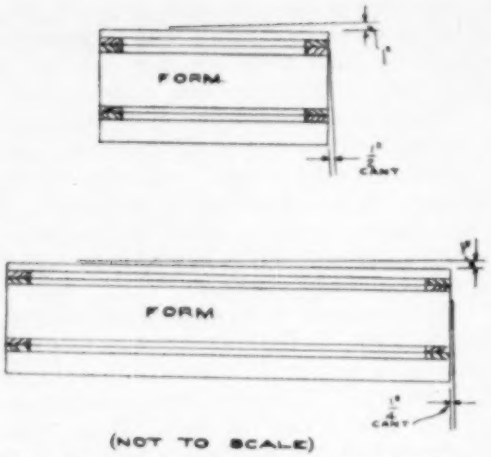


Fig. 28.—Showing Cause of Binding of Form to Wall.

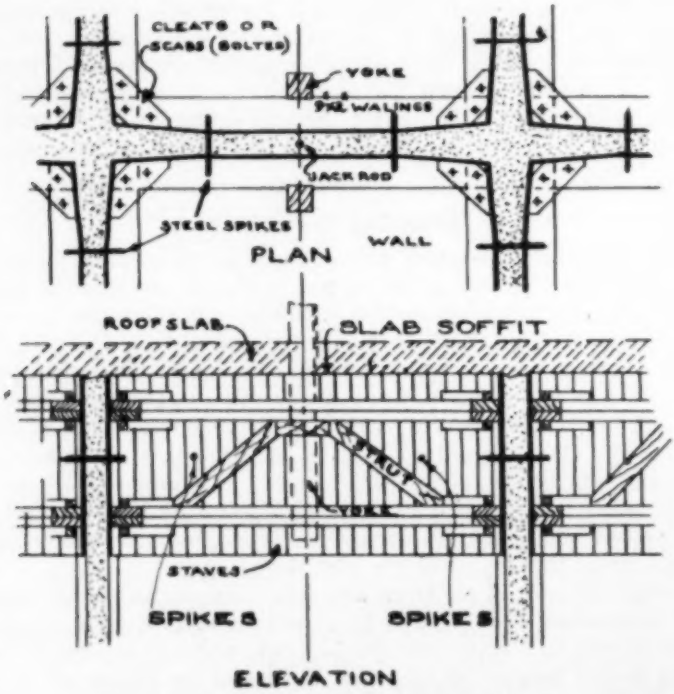


Fig. 29.—Deck used as Centering for Roof.

through with the eye from one to any other two cross-trees, the level of each yoke, and consequently of the forms, can be seen at a glance. Whenever there is a disparity of ¾ in. or more a general levelling must be undertaken, even if this is done between shifts.

It is advantageous to keep the outside corners of the forms a little high rather than low, and even if the four corners are, say, ¼ in. higher than the rest of the forms this will be "lost" in most cases before the next levelling takes place since the corners tend to drag behind the rest of the forms. Constant levelling is necessary to ensure that the forms do not bind on the walls. This is much more serious in the case of short forms than long ones. Unless the forms for short walls are kept constantly level, binding and consequent jamming of the forms will take place. This is seen from an example (*Fig.* 28). If a form is 1 in. out of level in a span of 8 ft. the horizontal movement of the form from the vertical will be ¼ in. for a form 4 ft. high. If the "draw" of the form is only ¾ in., the form will certainly jam. In the case of a form twice as long, the horizontal displacement will be ¼ in. and jamming will probably not occur because the "draw" will not be exceeded. Although the jack-men follow around the deck in a constant wave, each man having his own set of jacks, and although they may try to give each jack the same part of a turn, individual differences are inevitable as some men may give the jacks an extra part of a turn in making several turns.

Construction of Roof and Dismantling Forms.

In addition to providing a ready means of access to all parts of the work, the deck serves as shuttering to the soffit of the roof slab. At the roof level it is necessary to spike the forms in position, and to do this the staves are drilled to accommodate mild steel spikes (*Fig.* 29) which are usually ¾ in. diameter. To facilitate the withdrawal of the spikes at a later stage, they are wrapped in greased paper. When the spikes have been driven through the holes in the shutters, the yokes are dismantled and removed. In *Fig.* 29 the roof slab is shown completed, and after the concrete has hardened the forms are dismantled. There is no difficulty about this as the spikes hold each side of the forms firmly when the opposite side is taken away. The first operation is to remove the deck, then the wall forms are taken down separately and lowered through the bins and taken through the opening in the hopper-bottom if it is large enough. To provide a working scaffold for the men dismantling the forms, scaffold boards are placed across slings attached to the roof slab through holes formed in the slab for this purpose.

(To be continued.)

The Reconstruction of a Bomb-damaged Reinforced Concrete Factory.

DURING the war a large reinforced concrete factory in Lavington Street, Southwark, was damaged by a bomb. The building measures 220 ft. by 90 ft. and comprises a basement, ground floor, and three upper floors. The ground floor was almost entirely destroyed, the beams and slabs being forced downwards. About two-thirds of the first floor were destroyed, but only two small bays of the second floor. The three main internal columns were badly cracked from below the ground floor to just below the second floor. One of these columns was in a dangerous state and was cut away and replaced by a brick pier as a first-aid repair. All the external columns at the rear and side of the building, and the beams between them, were badly cracked. The third floor was only slightly damaged, but the roof covering and panel walls were blown away.

About half the building was rendered unusable by the damage sustained.

The building has been repaired by cutting away defective reinforced concrete and replacing with new. The defective ground-floor slabs and heavily-reinforced beams were cut away, the original reinforcement in the beams being cut off about 3 ft. from the face of each column (*Fig.* 1) to allow for bonding the new beams to the old columns. To replace the defective columns, a timber tower (*Fig.* 2) was erected on the basement floor to relieve each column in turn of its load. The tower, the calculated load on which was 80 tons, consisted of four legs built up from three 9-in. by 3-in. timbers bolted together with ¾-in. bolts at 18-in. centres.

Fig. 1.—Ground Floor Removed.

Each leg was placed under one of the floor beams supported by the column at the level of the second floor. The legs rested on folding wedges and timber sills, and were braced horizontally and diagonally by 6-in. by 3-in. timbers attached by bolts. The old column was then cut away and new reinforcement and shuttering fixed in position as seen in *Fig.* 2. Pockets formed in the new concrete (*Fig.* 3) and projecting reinforcement provided supports for the new beams constructed later. The columns at the rear of the building were dealt with in a similar manner except that a triangular tower was erected around each column, two of the

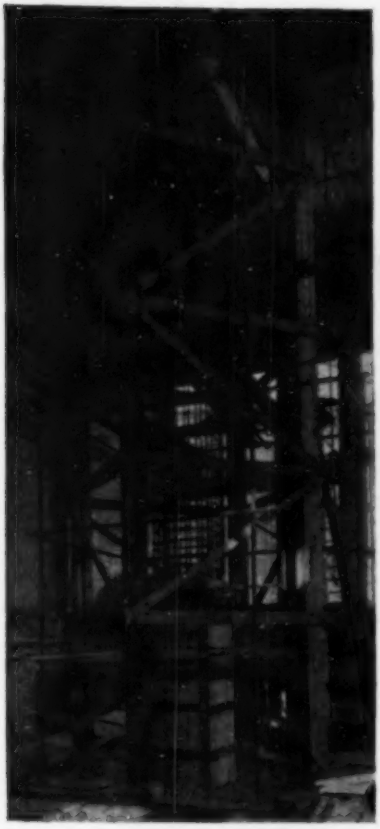


Fig. 2.—Construction of New Column.

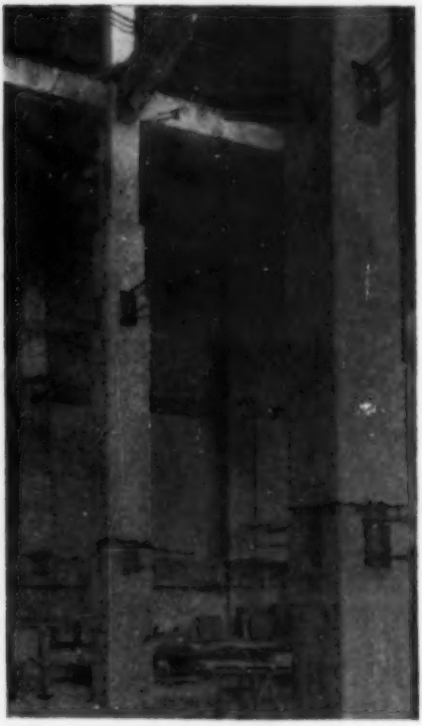


Fig. 3.—Repaired Columns.

Fig. 4.—Pockets cut in Column.

legs bearing on the wall-beams of the ground floor, the third bearing on the basement floor. Three towers were erected at the same time and were tied together by 9-in. by 3-in. timbers placed horizontally and bolted to the 9-in. square legs. The mixture of the concrete in the new columns is 1 cwt. of rapid-hardening Portland cement, 2 cu. ft. of sand, and 4 cu. ft. of ¾-in. graded ballast. The average crushing strength of the concrete at 28 days was 5500 lb. per square inch.

Where slight damage only had occurred at the corners of the columns and beams, and where the core of these members was sound, repairs were carried out by keying and applying new surfaces by cement-gun.

The part of the first floor which was not damaged is retained, but where the defective concrete had to be cut away the beams were supported temporarily on timber dead-shores. *Fig.* 5 shows part of the first floor with two main beams cut away and the ends of the secondary beams supported on struts bearing on the basement floor. The double 9-in. square timber shores were arranged to clear the secondary beams to be constructed at ground-floor level. When all the columns had been repaired the reinforcement for the main and secondary beams was fixed. The main beams are designed as freely-supported and are supported in pockets formed in the new columns or cut in the old columns. *Fig.* 4 shows the pockets

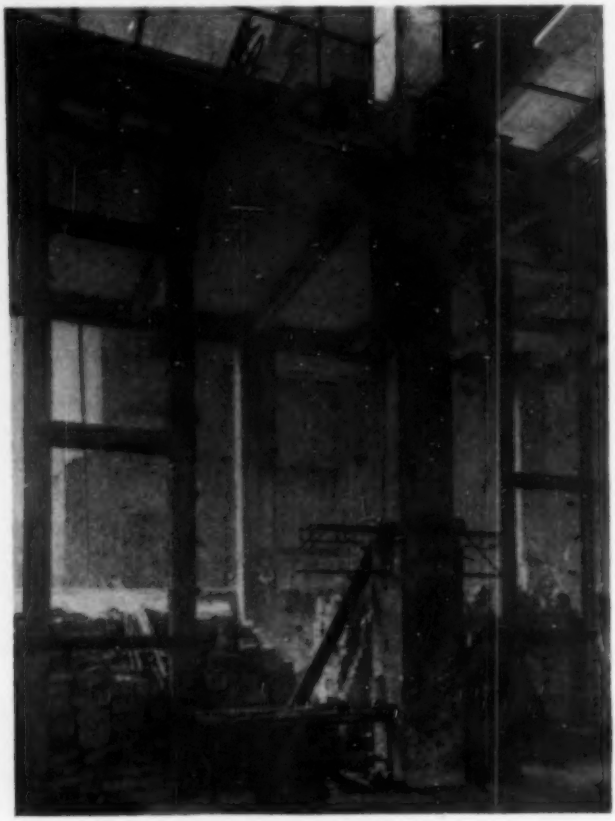


Fig. 5.—Timber Struts supporting Secondary Beams.

Fig. 6.—Shuttering and Reinforcement for Ground Floor.

cut in one of the old columns, the bars projecting from which were cut away to clear the new reinforcement. As the new reinforcement is so heavy (1¾-in. bars are provided in the main beams), it was necessary to fix the bars for the beams before the shuttering was erected. Steel plates were used for the shuttering for the slabs and secondary beams, and timber shuttering for the sides of the main beams. *Fig.* 6 shows three bays of shuttering in position. Rapid-hardening Portland cement was used for the concrete in the floors in order to expedite the release of the shuttering.

The architect for the reconstruction is Mr. T. F. Ford, F.R.I.B.A., the reinforced concrete was designed by Structural Steelcrete, Ltd., and the work was carried out by Messrs. Galbraith Brothers, Ltd.

Concreting a Dam in 50-ft. Lifts.

In view of the small depths of concrete commonly placed at one time in the construction of massive structures such as dams, it is interesting to learn that in the construction of some concrete dams in Ontario lifts up to 50 ft. are being concreted. A description of the construction of these dams is given in the Journal of the American Concrete Institute for April, 1950.

For the main dam, concrete was transported from the mixers on two 30-in. belt conveyors supported on bridges and trestles built of Bailey bridge trusses. The concrete was discharged into steel chutes supported on the shuttering and feeding into flexible trunks through which the concrete dropped vertically to the point of deposit. Up to 46,000 cu. yd. of concrete was placed monthly by this means, and daily amounts have exceeded 3,000 cu. yd. Parts of the dam up to 50 ft. high have been regularly placed in one continuous operation. With vertical joints at about 40 ft. intervals, 8,000 cu. yd. to 12,000 cu. yd. of concrete are required in such an operation. In the case of smaller dams not exceeding 60 ft. high, concreting was continuous from foundation to top. The cement was ordinary Portland cement but more coarsely ground than is usual. The largest size of aggregate was 3 in.

The object of the high lifts was to avoid the deterioration that is common at the horizontal construction joints in hydraulic structures. Other dams have been built in a similar manner in the past twenty years by the hydroelectric authority of Ontario, and examination has shown that no longitudinal cracking is evident, although it is recognised that the temperatures inside the dams are very high and will remain so probably for several years.

Composite Reinforced Concrete and Glass Construction.

At a recent meeting of the Reinforced Concrete Association, Mr. H. Wingrave Newell read a paper on composite reinforced concrete and glass. The components of this construction are reinforced concrete ribs between which glass lenses are inserted at the time of casting the concrete, and it is assumed that the glass and concrete act together in resisting forces and bending moments. Tests between the serrated edges of the lenses and the concrete, and in arch roofs the line of thrust is more easily maintained within the core or middle-third. The British method gives a lighter slab, does not induce tensile stresses in the glass (the tensile resistance of glass is very variable), and gives less marked variations of light and shade on the underside of the roof.

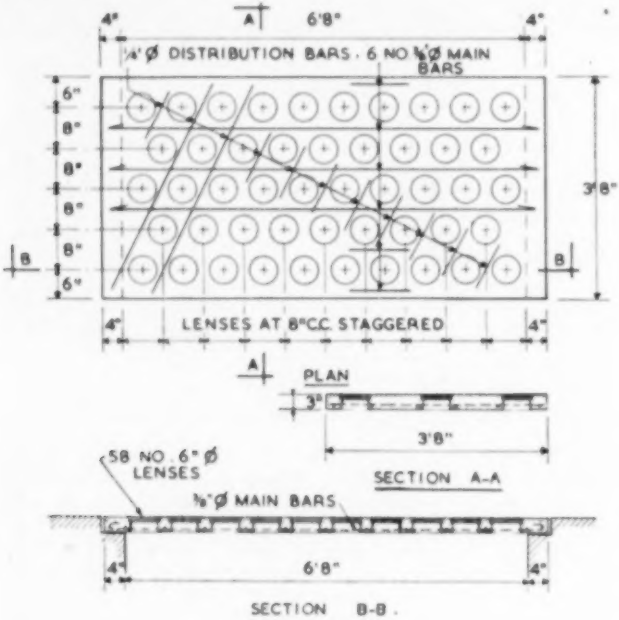


Fig. 1.

made on the Continent and in this country have proved this assumption to be justified.

British practice is to use principally shallow square or round lenses set in a reinforced concrete grid, the ribs projecting below the underside of the lenses, which rarely extend below the neutral axis of the composite construction and are used to resist compression. On the Continent it is common for the lenses to be of the same depth as the ribs, advantages of this method being that the moulds are cheaper, there is a better bond

There are two methods of design which assume that the glass and concrete act together. In one method it is assumed that the glass is stressed to the same degree as the adjacent concrete, that is the composite construction is considered to be the same as if it were entirely of reinforced concrete; this appears to be a very safe method of design. In the second method it is assumed that the strain on the glass is the same as that on the adjacent concrete. The modulus of elasticity of glass in compression, which is practically constant, is about

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10×10^6 lb. per square inch, that is about one-third that of steel and about five times that of concrete. The compressive strength of pressed glass lenses is considerably greater than that of concrete and exceeds 24,000 lb. per square inch. Therefore the compressive stress in the glass is generally very small compared with its strength. The coefficient of linear thermal expansion of the soda-lime glass used in pressed lenses is from 50×10^{-7} to 56×10^{-7} per deg. F. By using $\frac{1}{4}$-in. granite aggregate, the coefficient of 1 : 1 : 2 concrete having a water-cement ratio of 0·5 is from 60×10^{-7} to 66×10^{-7} per deg. F. Ordinary Portland cement and graded river sand are used, and the concrete gives a highly satisfactory solution of the problems of strength, watertightness, shrinkage, and expansion.

Example of Design.

The author has sent us the following numerical example of a slab designed in accordance with the second of the two methods described.

The problem is to determine the stresses in the glass, concrete, and reinforcement in the slab in *Fig. 1*, which is 3 in. thick and contains 58 pressed-glass lenses, each of which is 6 in. diameter and $\frac{3}{4}$ in. thick. The main reinforcement comprises six $\frac{3}{8}$-in. mild-steel bars (0·66 sq. in.) and has $\frac{1}{2}$ in. cover of concrete. The effective depth is 2·43 in. The superimposed load is 40 lb. per square foot and the dead weight of the slab is assumed to be 22 lb. per square foot. The total load is therefore

$$6{\cdot}67 \times 3{\cdot}67 \times 62 \text{ lb.} = 1364 \text{ lb.},$$

and the maximum bending moment is

$$1364 \times 7 \times 12 \times 0{\cdot}125 = 14{,}332 \text{ in.-lb.}$$

The average moduli of elasticity (in lb. per square inch) of concrete, glass, and steel are

Concrete : $E_c = 2 \times 10^6$.
Glass : $E_g = 10 \times 10^6$.
Steel : $E_s = 30 \times 10^6$.

The modular ratios are

Glass and concrete : $\frac{E_g}{E_c} = 5$.

Steel and concrete : $\frac{E_s}{E_c} = 15$.

The heterogeneous compressive area of the slab of composite reinforced concrete and glass construction may be replaced

by an equivalent homogeneous area of concrete in which the glass is replaced by five times its area of concrete. At the section where the maximum bending moment occurs there are two lenses. The net width of the concrete is 44 in. $-$ (2 $\times$ 6 in.) = 32 in., and the width of the glass is 2 $\times$ 6 in. = 12 in. The width of the equivalent concrete compressive flange is

$$32 \text{ in.} + (5 \times 12 \text{ in.}) = 92 \text{ in.}$$

The depth to the neutral axis from the top of the slab is

$$\frac{15 \times 0{\cdot}66}{92}\left(\sqrt{1 + \frac{2 \times 92 \times 2{\cdot}31}{15 \times 0{\cdot}66}} - 1\right) = 0{\cdot}606 \text{ in.};$$

the neutral axis therefore lies within the thickness of the glass. The lever arm of the moment of resistance is

$$2{\cdot}31 - \frac{0{\cdot}606}{3} = 2{\cdot}11 \text{ in.}$$

The maximum compressive stress in the concrete is

$$\frac{2 \times 14{,}332}{92 \times 0{\cdot}606 \times 2{\cdot}11} = 244 \text{ lb. per square inch.}$$

The maximum compressive stress in the glass is therefore 5 $\times$ 244 = 1220 lb. per square inch. The tensile stress in the reinforcement is

$$\frac{14{,}332}{2{\cdot}11 \times 0{\cdot}66} = 10{,}300 \text{ lb. per square inch.}$$

The critical section for resistance to shearing force is 10 in. from either support, where there are five lenses cut by a cross section and where the net width of the concrete is 44 in. $-$ (5 $\times$ 6 in.) = 14 in. The shearing force at this section is

$$\frac{1364 \times 30}{2 \times 40} = 524 \text{ lb.},$$

and the shearing stress is

$$\frac{524}{2{\cdot}11 \times 14} = 18 \text{ lb. per square inch.}$$

Training Courses in Concrete.

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Effect of Ice on Structures.*

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An ice sheet expands and contracts with changes of temperature and thereby exerts considerable force on a structure in contact with it. The average coefficient of linear expansion for ice from 0 to 32 deg. F. is about 0·00003 per degree F. The actual expansion is about half of that indicated by the coefficient if applied to the temperature of the air, because the temperature of the lower surface of the ice sheet is at about the freezing point of the water beneath it. The pressure which the ice exerts cannot exceed its crushing strength. Under slowly applied loads ice appears to have a crushing strength, which varies with the temperature, from about 400 lb. per square inch at freezing point to twice this strength at 0 deg. F.

The actual pressure on a dam depends on many factors. If the shores of the reservoir are of yielding material, they may absorb the expansion and little thrust will be exerted on the dam. This is also true if the opposite shore is flat enough to permit the ice sheet to override the bank. If the ice sheet is large it is likely to buckle and form pressure ridges, which relieve the pressure. If the ice sheet is free to expand parallel to the dam, the pressure exerted on the dam will be less than if the sheet is restrained in this direc-

* Abstracted by permission of McGraw-Hill Book Co. from "Applied Hydrology," by R. K. Linsley, M. A. Kohler, and J. L. H. Paulhus (1949). This book was reviewed in this journal for January, 1950.

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tion. Sloping the upstream face of a dam might be expected to relieve the thrust from the ice, but since ice may adhere to concrete with a bond stress of about 200 lb. per square inch, the increased area exposed to adhesion by the sloping face may more than overcome the tendency of the ice sheet to slide up the sloping face.

The expansion of ice due to a rise of temperature is augmented by a ratchet action resulting from cracking and re-freezing. When an ice sheet contracts because of a drop in temperature, it contracts most at the upper surface. Because the tensile strength of ice is low (about 200 lb. per square inch), cracks may occur in the upper surface. If the cracks fill with water which subsequently freezes, the ice sheet is lengthened. Similarly, contraction may cause the sheet to crack near the shore, the cracks immediately filling with water, which freezes and, with

TABLE I.—THRUST OF ICE (LB. PER LINEAR FOOT).

Thickness of ice sheet (ft.)	No solar radiation			With solar radiation (Latitude 40 deg.)		
	Rate of rise of temperature (deg. F. per hour)					
	5	10	15	5	10	15
0·5	1100	1600	3300	1500	3600	6,000
1·0	2200	2800	4800	2700	5000	8,000
2·0	4200	5100	7600	3800	7300	10,300
3·0	6200	7300	9700	7000	9100	12,300

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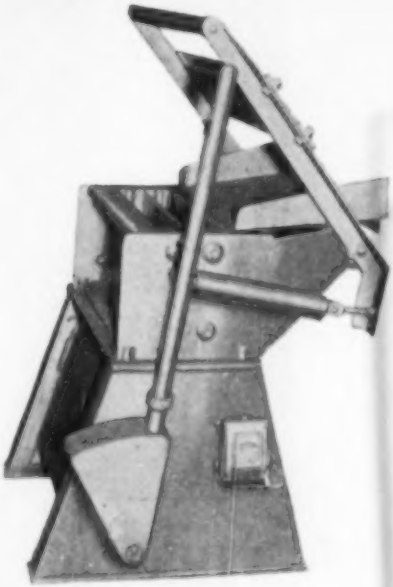

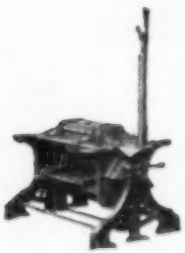

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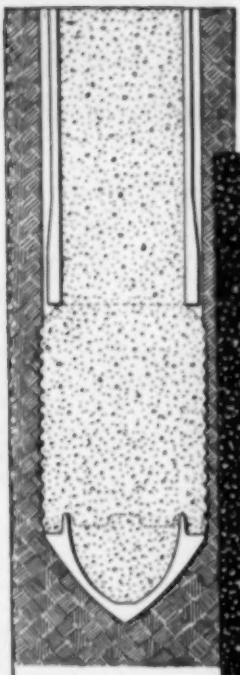

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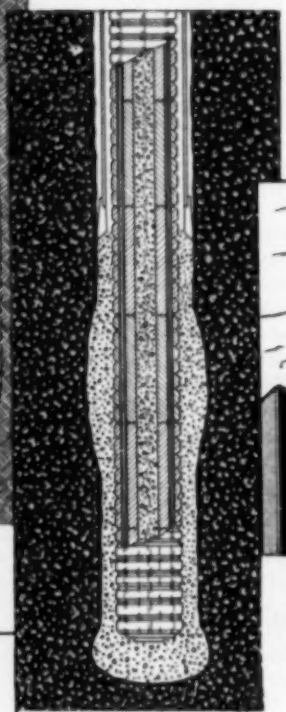

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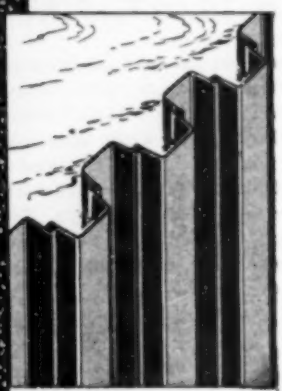

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the next expansion, the shore ice is pushed farther inland. Ridges of soil and gravel sometimes form along river banks as a result of this action.

Many dams in the northern United States have been designed for pressures of 30,000 lb. to 60,000 lb. per foot applied at the highest winter water-level, but evidence indicates that these pressures are too high. Because of the plastic property of ice under stress, large pressures can be developed only with rapid changes in temperature. Probable ice thrusts for various thicknesses of ice and rates of change of temperature, with and without solar radiation, and assuming no lateral restraint, are given by Mr. E. Rose in the Transactions of the American Society of Civil Engineers (vol. 112, 1947, pp. 871–900). *Table* No. I is based on the data given by Mr. Rose. The temperature is assumed to rise at the rate indicated from − 40 deg. F. to + 32 deg. F. and to remain constant until the ice sheet is isothermal. For complete lateral restraint the pressures should be multiplied by 1·575, assuming Poisson's ratio to be 0·365. The absorption of solar radiation results in a further increase of temperature in addition to the effect of change of air temperature.

Artificial heating to prevent the formation of ice adjacent to a dam and to maintain a channel of open water is the best protection against ice thrust. Compressed air released from orifices 10 ft. to 20 ft. below the water surface agitates the water and carries warm water up with the bubbles, and is an effective and economical means of maintaining a channel. Fluctuation of the water surface tends to break the ice and reduce the pressure.

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SITUATION VACANT. Men who are experienced in the design of reinforced concrete structures and who hold a University degree and/or A.M.Inst.C.E. are invited to apply for a post we have vacant at our Manchester design office. Write, giving full details of experience and salary required, to the SECRETARY, BOX 2364, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

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SITUATION VACANT. Reinforced concrete designer with wide experience in all types of reinforced concrete structures required for project section of a London firm of reinforced concrete engineers. Salary up to £900 per annum. Write, giving full details of experience, training and age. Applications will be treated in confidence. BOX 2373, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATION VACANT. We are requiring a man who is thoroughly experienced in the design of reinforced concrete structures. He should hold a University degree and/or A.M.Inst.C.E. The vacancy occurs in our Glasgow design office. Write in confidence to the Managing Director, BOX 2363, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, Westminster, S.W.1.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers with specialist experience for their Stafford, Dublin and London offices. Five-day week, staff pension scheme, social club and canteen. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

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Continued on page 224.

MISCELLANEOUS ADVERTISEMENTS

(Continued from page 223)

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